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# CLOVER FORK TUNNEL DIVERSION PROJECT HARLAN, KENTUCKY

## Hydraulic Model Investigation

by

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<p>A tunnel diversion project has been authorized that will divert the full flow of the Clover Fork near Harlan, KY, to an outlet downstream of the city. The project has a diversion structure at the upstream end of the project that will direct the flow of the Clover Fork into four inverted U-shaped tunnels that average 1,936 ft in length. The narrow floodplain upstream of the project site is filled with debris that could potentially reach the entrance to the tunnels during the Standard Project Flood (SPF).</p> <p>A 1:30-scale physical model was constructed to assess the tunnel entrance for the improvement of hydrodynamic conditions and the mitigation of potential floating debris problems. A two-dimensional depth-averaged numerical hydrodynamic model was used in conjunction with the physical model to screen alternatives that would streamline the approach. The physical model investigated these alternatives for their capability to pass</p> <p style="text-align: right;">(Continued)</p>					
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19. ABSTRACT (Continued)

debris by reproducing the SPF hydrograph while simultaneously introducing predicted volume of scaled debris.

Features reproduced in the physical model included about 2,000 ft of the natural channel upstream of the tunnel entrance, the diversion structure, the proposed tunnel entrance, and 450 ft of the tunnels. The testing resulted in two entrance designs that not only passed the SPF flow with debris but also provided sufficient freeboard along the diversion structure.

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## PREFACE

The model investigation and analysis described herein were authorized by Headquarters, US Army Corps of Engineers (USACE), on 24 September 1987 at the request of the US Army Engineer District, Nashville (ORN). The model tests were conducted on an accelerated schedule at the request of ORN during the period May 1988 to December 1988 in the Hydraulics Laboratory of the US Army Engineer Waterways Experiment Station (WES).

Testing was conducted under the direction of Messrs. F. A. Herrmann, Jr., Chief of the Hydraulics Laboratory; R. A. Sager, Assistant Chief of the Hydraulics Laboratory; G. A. Pickering, Chief of Hydraulics Structures Division (HSD); and J. F. George, Chief of Locks and Conduits Branch (LCB), HSD. Physical model tests were conducted by Ms. Sandra K. Martin and Mr. Van Stewart, LCB. The computer code, STREMR, used in this study was written by Dr. Robert S. Bernard, Reservoir Water Quality Branch, HSD. Dr. Bernard's expertise and timely completion of the model tests were an integral part of the successful completion of this study. This report was prepared by Ms. Martin and edited by Ms. Marsha C. Gay, Information Technology Laboratory, WES.

Prior to model testing, Ms. Martin visited the proposed project site with ORN personnel in Harlan, KY, and vicinity to inventory debris in the basin. Visitors to WES during the model tests included Messrs. Sam Powell and Tom Munsey, USACE; Mr. Lynn Richardson, US Army Engineer Division, Ohio River; LTC William Allen, Deputy District Engineer, ORN; Messrs. Warren Bennett, Euclid Moore, Hank Phillips, Tom Allen, Jody Stanton, Ben Couch, and Marvin Simmons, ORN, and several representatives from private industry.

Acting Commander and Director of WES during preparation of this report was LTC Jack R. Stephens, EN. Technical Director was Dr. Robert W. Whalin.

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CONVERSION FACTORS, NON-SI TO SI (METRIC) UNITS  
OF MEASUREMENT

Non-SI units of measurement used in this report can be converted to SI  
(metric) units as follows:

<u>Multiply</u>	<u>By</u>	<u>To Obtain</u>
cubic feet	0.4535924	kilograms
degrees (angle)	0.01745329	radians
feet	0.3048	metres
miles (US statute)	1.609344	kilometres
square feet	0.09290304	square metres
square miles	1.609344	square kilometres

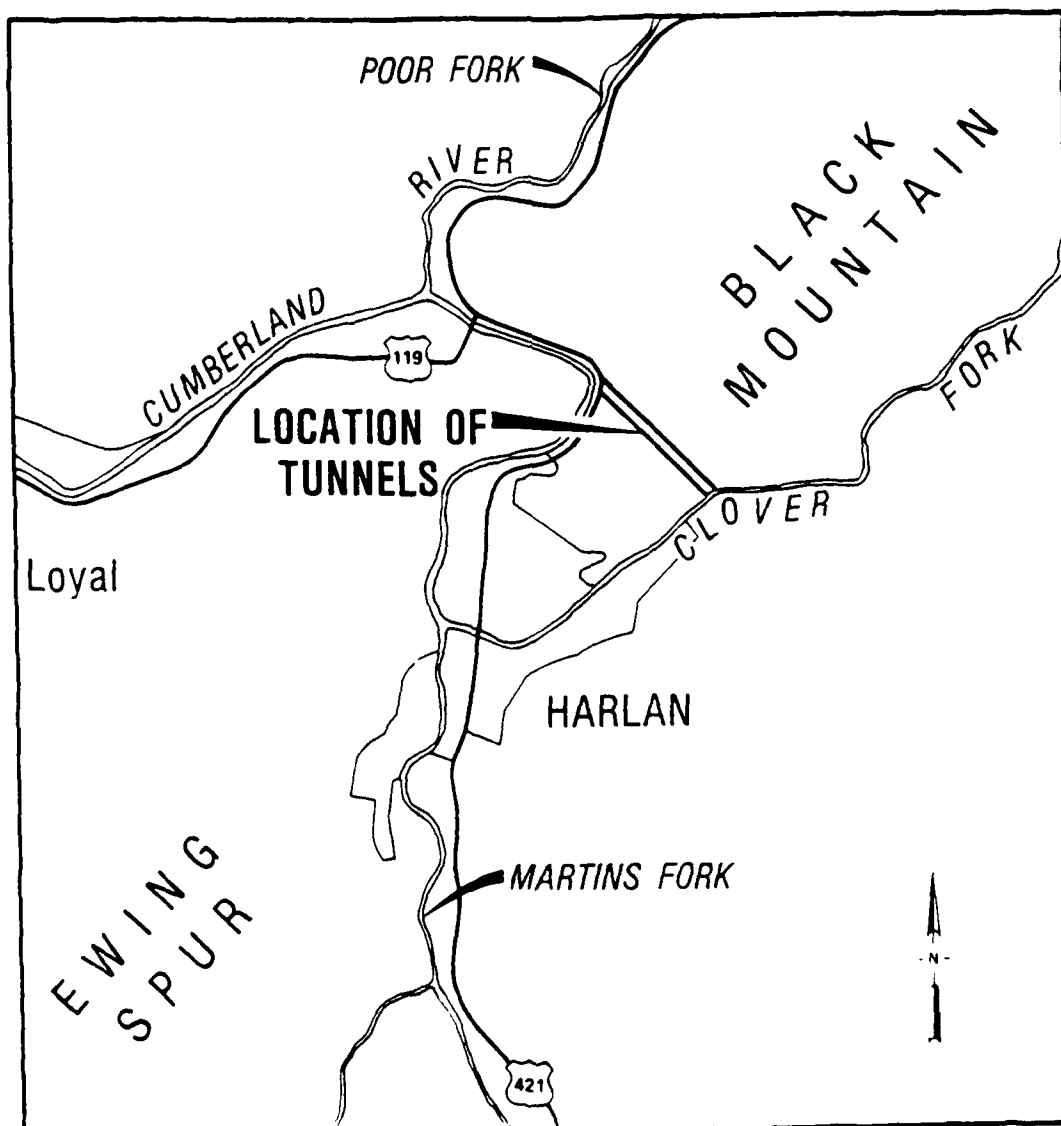
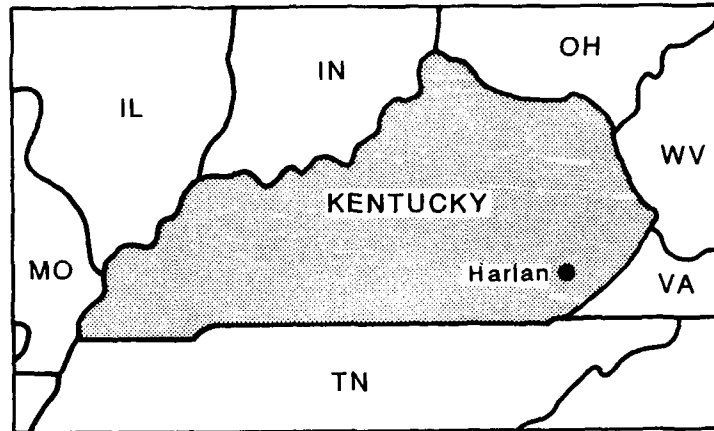


Figure 1. Location and vicinity map

CLOVER FORK TUNNEL DIVERSION PROJECT

HARLAN, KENTUCKY

Hydraulic Model Investigation

PART I: INTRODUCTION

The Prototype

1. Martins Fork joins Clover Fork in Harlan, KY, approximately 1 mile\* south of Clover Fork's confluence with Poor Fork and the Cumberland River (Figure 1). Historical flooding of the three forks can be characterized as fast rising (8-10 hours) and short lived in duration (approximately 24-30 hours). Both the Martins Fork and Clover Fork channels are approximately 100 ft wide, have stream slopes of about 11 ft per mile, and have banks 5 to 15 ft high. Clover Fork and Martins Fork drain approximately 104 and 117 square miles, respectively, of primarily forested land. Clover Fork's narrow floodplain, averaging approximately 1,000 ft in width, is cluttered with trash, dead brush, potentially unstable housing such as trailers, and stockpiles of logs from clearing operations in the area. The shallow channel banks show signs of instability and are lined with large trees that lean precariously toward the channel bed.

2. A tunnel diversion project is authorized that will provide Standard Project Flood (SPF) protection by diverting the full flow of Clover Fork around the Harlan central business district. A diversion structure will be constructed at the upstream end of the project area to direct the flow from Clover Fork into four tunnels that discharge downstream of Harlan (Plate 1). The size of the Harlan tunnels and the diversion levee height were designed to ensure SPF protection with an appropriate level of freeboard on the diversion structure. Each tunnel has an inverted U-shaped cross section, approximately 33 ft wide by 32 ft high. Separation between tunnels is approximately 35 ft. The tunnels, averaging 1,936 ft in length, are designed to pass a maximum SPF discharge of 52,800 cfs with an assumed percentage of debris blockage at the entrance. The proposed headwater elevation that determined the levee height

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\* A table of factors for converting non-SI units of measurement to SI (metric) units is found on page 3.

was calculated assuming an entrance loss coefficient due to blockage by debris.

#### Purpose and Scope of Model Investigation

3. One of the major project concerns was the potential retention of floating debris at the upstream face of the tunnels. Blockage at the portal face resulting from the accumulation of debris would raise the headwater in the approach area and thereby decrease the available freeboard on the diversion structure. The study was initiated to evaluate the adequacy of the proposed tunnel entrance to pass debris and to maintain the design freeboard, and/or to develop modifications that would not only improve the hydrodynamic conditions, but also mitigate potential debris problems at the upstream portal.

4. Both a physical and a mathematical model were used to evaluate the proposed entrance approach. The mathematical model was used in conjunction with the physical model to screen alternatives, while the physical model investigated these alternatives in detail for their ability to pass debris by simultaneously reproducing the SPF hydrograph while an expected volume of scaled debris was introduced. The main objectives of the study were to evaluate and improve the hydrodynamic design of the entrance approach, and to select a design configuration, especially regarding the tunnel piers and portal face, that minimizes head losses associated with debris.

## PART II: THE MODELS

### Physical Model

5. The 1:30-scale physical model reproduced approximately 2,000 ft of the natural channel upstream of the entrance, the diversion structure, the proposed channel approach, and 450 ft of the tunnels (Figure 2). All elements of the model except the tunnels were constructed by molding sand and cement mortar to sheet metal templates. The tunnels were constructed of transparent plastic for observation of water and debris flow conditions.

#### Model appurtenances

6. Water used in the operation of the model was supplied by a circulating system. Discharges were measured with a venturi meter installed in the flow line and were baffled before entering the model. Steel rails graded to specific elevations were placed along both sides of the model to serve as supports for measuring devices. Velocities were measured with an electronic velocity meter mounted to permit measurement of flow from any direction and at any depth. Water-surface elevations were measured with point gages, and pressures were measured with piezometers. Tailwater conditions were regulated by adjusting gates located at the downstream end of each tunnel and monitoring piezometers located on the floor of the tunnels. Different designs for SPF peak flow conditions and SPF hydrographs with debris were recorded photographically.

#### Scale relations

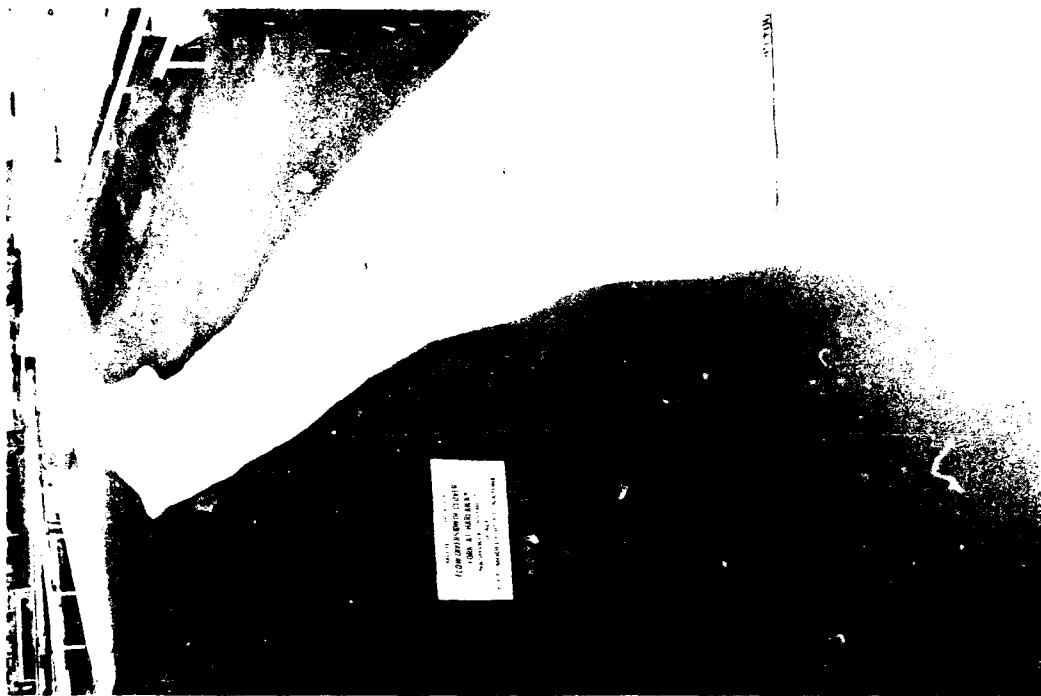
7. The equations of hydraulic similitude, based on the Froude criteria, were used to express mathematical relations between the dimensions and hydraulic quantities of the model and prototype. General relations for transferring model data to prototype equivalents are as follows:

<u>Characteristic</u>	<u>Dimension*</u>	<u>Scale Relations Model/Prototype</u>
Length	$L_r$	1:30
Area	$A_r = L_r^2$	1:900
Velocity	$V_r = L_r^{1/2}$	1:5.477
Time	$T_r = L_r^{1/2}$	1:5.477
Discharge	$Q_r = L_r^{5/2}$	1:4,929.5
Weight	$W_r = L_r^3$	1:27,000
Roughness coefficient	$N_r = L_r^{1/6}$	1:1.763

\* Dimensions are in terms of length



a. Looking upstream



b. Looking downstream

Figure 2. General view of the model

### Math Model

8. STREMR, the numerical model selected for this study, is a finite-difference computer code that uses the governing equations for depth-averaged incompressible flow in two dimensions (2-D). The model was developed by Dr. Robert S. Bernard for the Repair, Evaluation, Maintenance and Rehabilitation (REMR) Research Program sponsored by Headquarters, US Army Corps of Engineers (USACE) (Bernard 1987). Lateral boundaries and obstacles can have any shape, and bathymetry can vary with position. The code employs a predictor-corrector scheme to solve the discretized governing equations; resistance on the bottom is computed from Manning's equation. The flow may be either steady or time-dependent.

9. The fundamental assumption in the STREMR code is that the flow upstream of a hydraulic structure can be described by the depth-averaged equations of motion for an incompressible fluid. Additionally, it is assumed that the displacement of the free surface from its initial elevation is small; therefore, the computational free surface becomes a rigid lid. This assumption is exact at a Froude number equal to 0, but gradually breaks down as the Froude number approaches 1.

### PART III: TESTS AND RESULTS

10. Seven different approach designs were constructed in the physical model. To streamline the approach hydrodynamically, evaluate passage of debris, and assess entrance losses at the portal face, three kinds of tests were conducted. In the first kind of test, velocities and water-surface profiles were used to compare types 1-7 design approaches during the SPF peak flow. Secondly, the response of types 4-7 approaches to floating debris was tested by reproducing the SPF hydrograph while introducing debris. Finally, the coefficient for entrance loss was determined for variable blockage conditions on the type 6 design approach.

#### Approach Channel Designs

11. The seven different approach designs are described as follows:

- a. Type 1 design approach. Original design approach channel with the portal face having a vertical slope of 4V:1H without tunnel piers (Photo 1, Plate 2).
- b. Type 2 design approach. The same approach layout as the type 1 design approach except that the portal face was vertical, a 5-ft-radius bellmouth was installed on the tunnel openings, and three tunnel piers and two bridge piers were installed according to drawings provided by the US Army Engineer District (USAED), Nashville (Photo 2, Plate 3).
- c. Type 3 design approach. Same as the type 2 design approach except that the existing right bank approach wall was replaced with a curvilinear transition section (Photo 3, Plate 4).
- d. Type 4 design approach. The type 3 design approach was modified by lowering the noses on tunnel piers 1 and 3, removing the bridge piers on top of tunnel piers 1 and 3, and replacing them with one bridge pier located on the center of tunnel pier 2 (Photo 4, Plate 5).
- e. Type 5 design approach. The tops of the tunnel piers in the type 4 design approach at the portal face were raised to elevation 1210.0,\* and the sloping pier noses were extended up to this elevation (Photo 5, Plate 6).
- f. Type 6 design approach. The type 5 tunnel piers were completely removed and replaced with piers designed similar

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\* All elevations (el) cited herein are in feet referred to the National Geodetic Vertical Datum (NGVD).

to those in Engineer Manual (EM) 1110-2-1601 (Headquarters, USACE, 1970) (Photo 6, Plate 7). The 5-ft-radius bellmouth on each tunnel was replaced with a 10.5-ft bell.

- g. Type 7 design approach. The final pier design, furnished by Nashville District, consisted of 120-ft-long tunnel piers with a sloping nose that had an 8-ft radius (Photo 7, Plate 8). Since the top of the piers extended above the headwater elevation along the roadway alignment, the bridge pier was not necessary.

In types 2-7 designs, a warped transition section was needed to tie the tunnel piers and the bellmouth smoothly into the tunnel walls.

12. The bridge and tunnel pier designs in types 2, 3, 4, and 5 were provided by the Nashville District. The bellmouth entrance to the tunnels, the curvilinear right bank wall, and the tunnel pier design in type 6 were recommended for testing as modifications for the improvement of hydraulic conditions. The bridge piers were needed to support a roadway proposed to cross the tunnel piers at an alignment specified in the Harlan Diversion Project Feature Design Memorandum (USAED, Nashville, 1988). Type 7 tunnel piers were selected for testing from three options designed by the staff at Nashville District that potentially satisfied both hydraulic and geotechnical attributes.

#### Model Data and Observations

13. Initially, there was some concern that the abrupt transition from the natural channel into the tunnels would skew the flow distribution down each tunnel, thereby decreasing the total capacity of the tunnels. The tunnels were sized assuming an even distribution of flow down each tunnel and a level headwater across the portal face. Surface currents, as exhibited by confetti, displayed a tendency to pull to the left bank wall. Therefore, velocities were taken to ensure that placement and shape of piers did not drastically alter the distribution of the flow down each tunnel. Water-surface elevations at the portal face were taken to evaluate the hydraulic efficiency of the various entrance conditions. Results of these tests are summarized in Table 1.

14. Velocities and water-surface profiles were recorded for types 1-7 design approaches during steady-state flow conditions. The peak inflow of

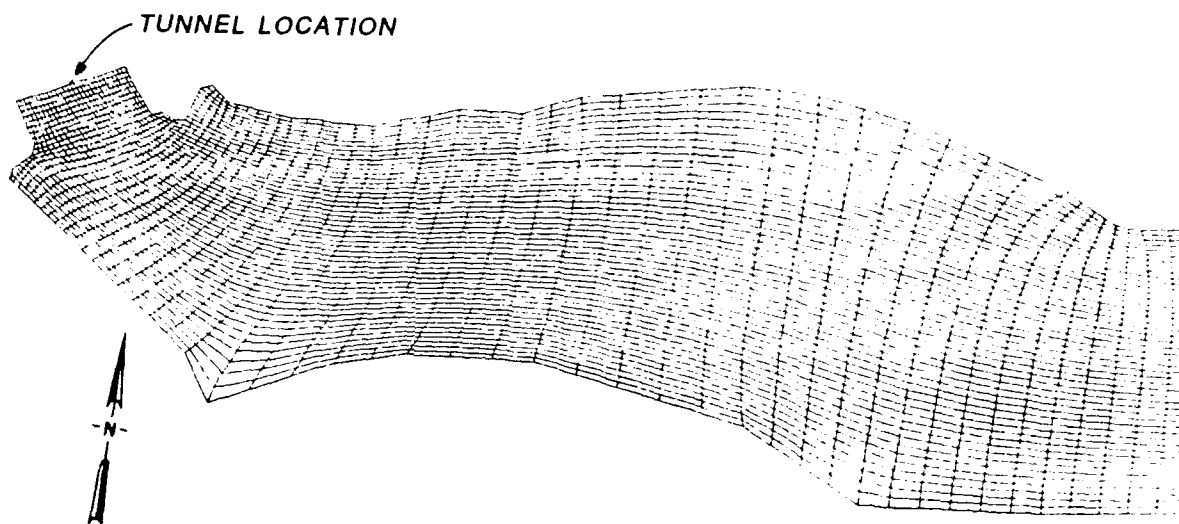
52,800 cfs was reproduced with a corresponding peak tailwater elevation of 1195.6 at the downstream tunnel outlet. The tailwater was reproduced by controlling gates at the outlet to each tunnel in the model until piezometers located 420 ft (prototype) downstream from the tunnel entrance produced a point on the hydraulic grade line equivalent to el 1198.7 at that station. The velocities were taken at five elevations for stations 2.5 ft, 45 ft, and 90 ft upstream of the portal face and along the center line of the tunnels. At all other stations, velocities were taken at near-bottom, near-top, and middepth elevations laterally across the section. Photos 8 and 9 reflect current patterns for type 1, the original design, and type 7, the selected design, respectively. These photographs also indicate the close correlation between STREMR results discussed in the following section and physical model observations. Velocities measured with the type 1 design approach and type 7 design approach are shown in Plates 9 and 10, respectively.

#### Numerical Model Tests

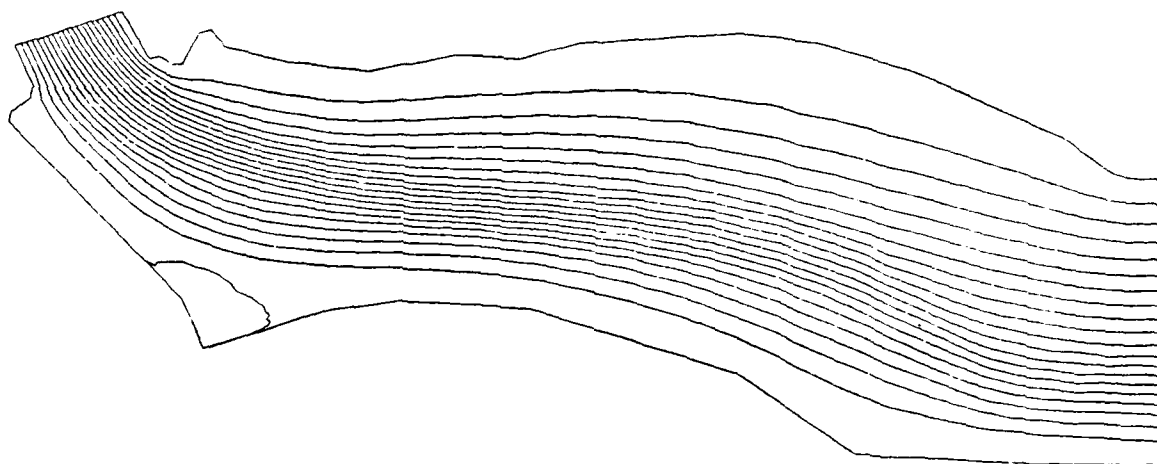
15. The STREMR 2-D model was used simultaneously with the physical model to assess the entrance conditions to the proposed tunnel site. The model grid was originally developed for a far-field approach to assess general flow patterns in the natural channel for approximately 3,000 ft upstream of the tunnel entrance. Figure 3 shows the general layout grid, streamlines, and velocity vectors.

16. The grid was then refined for an intermediate range covering approximately 1,000 ft upstream of the portal face. Model tests performed with this grid verified the location of local eddies in the physical model, identified flow separation points, and approximated the direction and relative magnitude of the velocity vectors observed in the physical model. As an example of the correlation between models, Figure 4 displays velocity vectors measured in the physical model and those calculated from an intermediate-range grid of the type 2 design approach. The type 3 design approach, in which the right bank approach wall was modified, was recommended as a result of these tests (Plates 11-13).

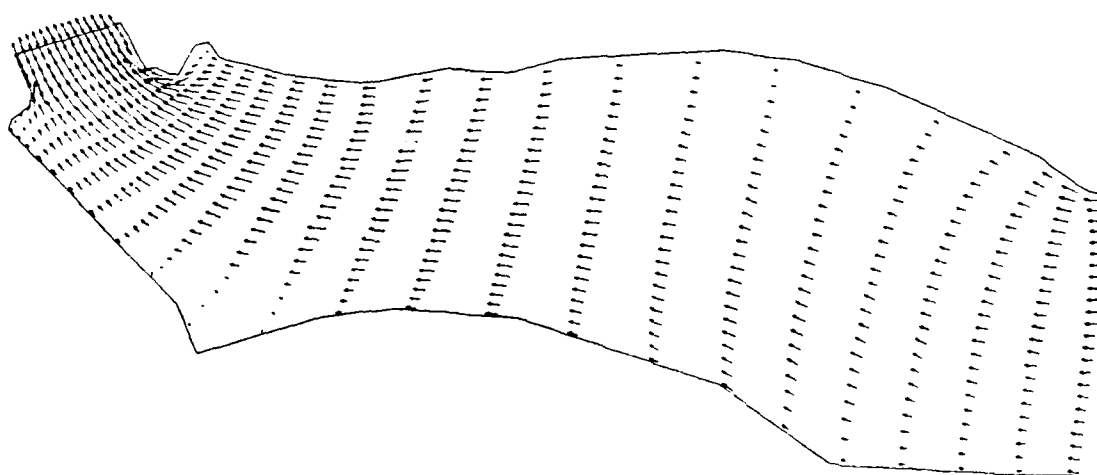
17. Finally, two grids were developed for close-approach calculations with the type 4 design approach piers in place: one with the original right bank approach and one with the type 3 curvilinear right bank approach. As can



a. Grid

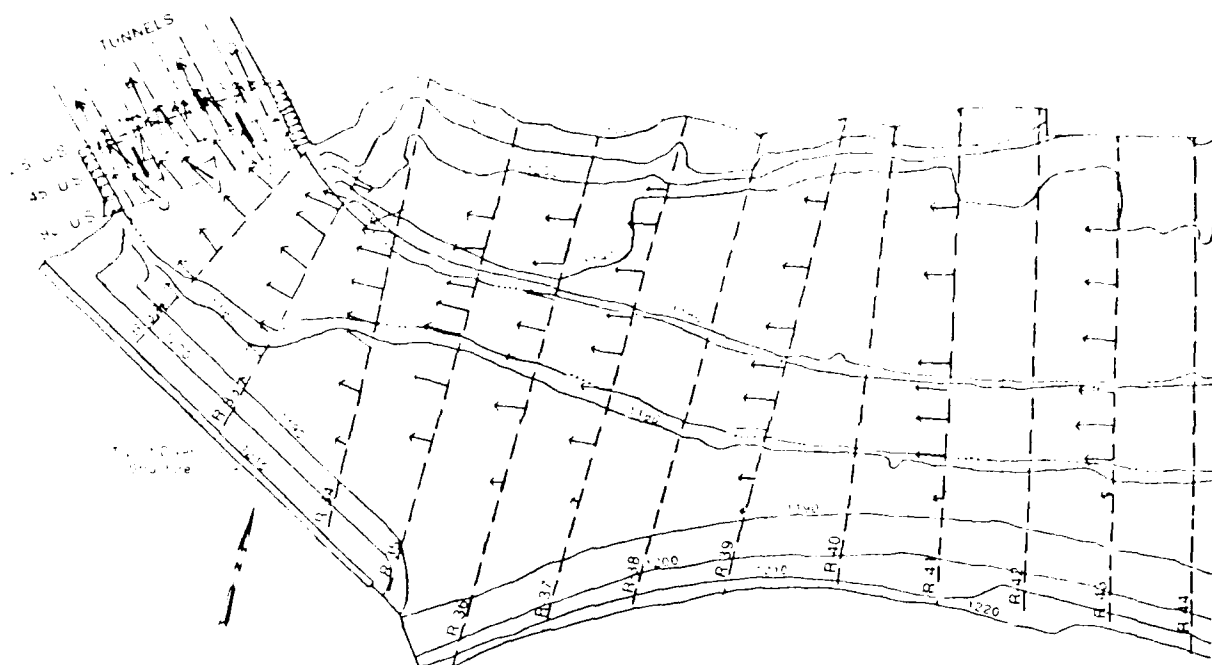


b. Streamlines

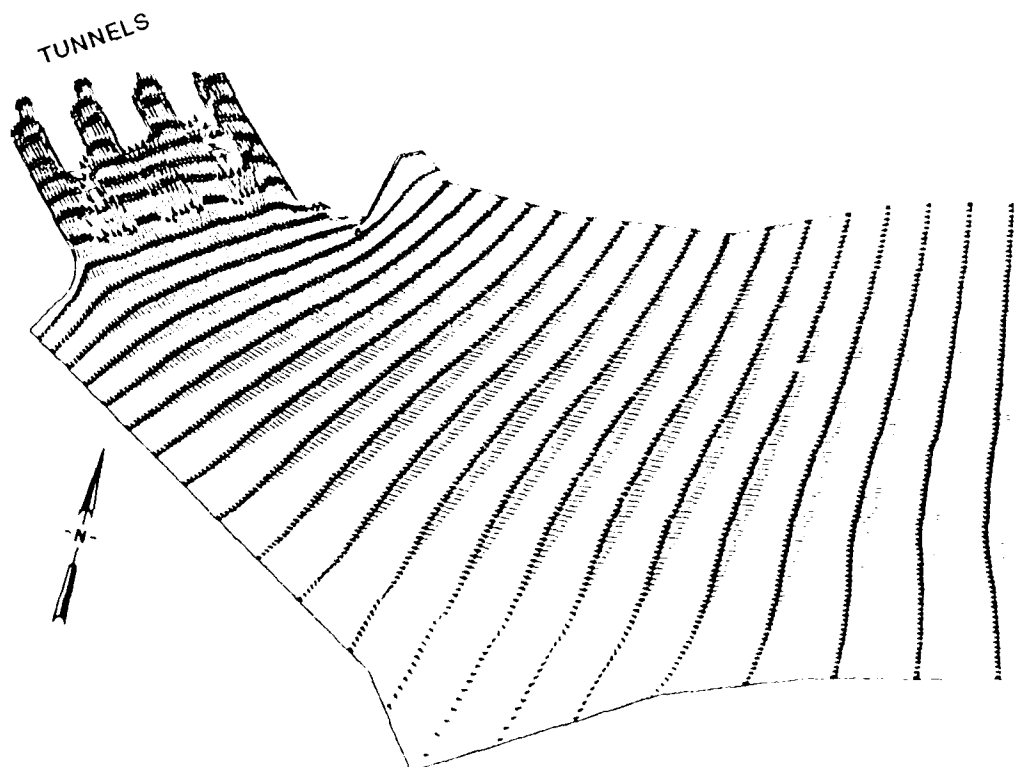


c. Velocity vectors

Figure 3. STREMR computations for far-field approach



a. Physical model depth-averaged velocity data



b. STREMR computed velocities

Figure 4. Velocity vectors, type 2 design approach

be seen by the streamlines and velocity vectors shown in Plates 14-16, smoothing the right bank wall removed the separation point and eddy observed by the original right bank wall. In the physical model, the flow separation was eliminated as was the eddy; however, some local turbulence occurred at the bank-water interface. While the correlation between the models was good, three-dimensional effects cannot be reproduced in a 2-D model. The mathematical model also supported conclusions drawn from the velocity data in the physical model, that each tunnel carries approximately one-quarter of the total discharge.

### Debris Tests

#### Volume

18. Since the major concern of the model study was to assess the ability of the various designs to pass debris through the tunnels and maximize available freeboard on the diversion structure, a testing methodology was developed in which a specific volume of scaled debris was introduced into the model during the SPF hydrograph. The volume of debris expected to reach the upstream entrance to the tunnels during a storm of this magnitude was determined by Nashville District. The estimated volume of debris was 1,925,000 ft<sup>3</sup>, or approximately 72 ft<sup>3</sup> in the model. This estimate was based on information obtained from a field investigation in April 1988, available literature concerning inventory methodologies (McFadden and Stallion 1976), and photographs taken during historical flood events in the region.

#### Mixture

19. The debris accessible to transport in the Clover Fork floodplain is a heterogeneous mixture having various shapes, specific weights, and sizes. For purposes of quantifying the debris, it was broken into several categories: large trees, small trees, branches and twigs, housing, and housing parts. A worst-case distribution by volume was to consist of 70 percent large objects (defined as those with nominal dimensions exceeding 35 ft) including intact houses and trailers. The remaining 30 percent was to include smaller objects such as broken trailers, twigs, and branches. A sensitivity test using this debris scenario on the type 4 design approach during the SPF hydrograph resulted in overtopping of the diversion structure. The elevation of the structure in the model is 1210.0. The final design top elevation of the

diversion structure is approximately 1212.0. Skewing the distribution of debris to the larger objects was felt to be inappropriate by personnel from USACE, the US Army Engineer Division, Ohio River, Nashville District, and the US Army Engineer Waterways Experiment Station. A more reasonable distribution was agreed upon in which 80 percent of the total volume consisted of smaller objects including broken pieces of trailers and housing of variable buoyancy, as well as twigs and branches. The remaining 20 percent included trees from 35 to 90 ft tall, some of which were fabricated to have nonbuoyant root systems. Table 2 lists the distribution of the debris.

#### The hydrograph with debris

20. The inflowing SPF hydrograph used during all tests began 3 hours prior to the peak and continued through the peak for 3 hours on the recession limb for a total of approximately 6 hours, prototype. Consequently, the beginning flow was 47,000 cfs, rising to a peak of 52,650 cfs, and receding to 43,500 cfs. Simultaneously, the tailwater hydrograph was reproduced by operation of the downstream gates on the tunnels. Both the inflowing hydrograph and the tailwater hydrograph were provided by Nashville District (Figure 5). The debris was introduced evenly throughout the hydrograph except

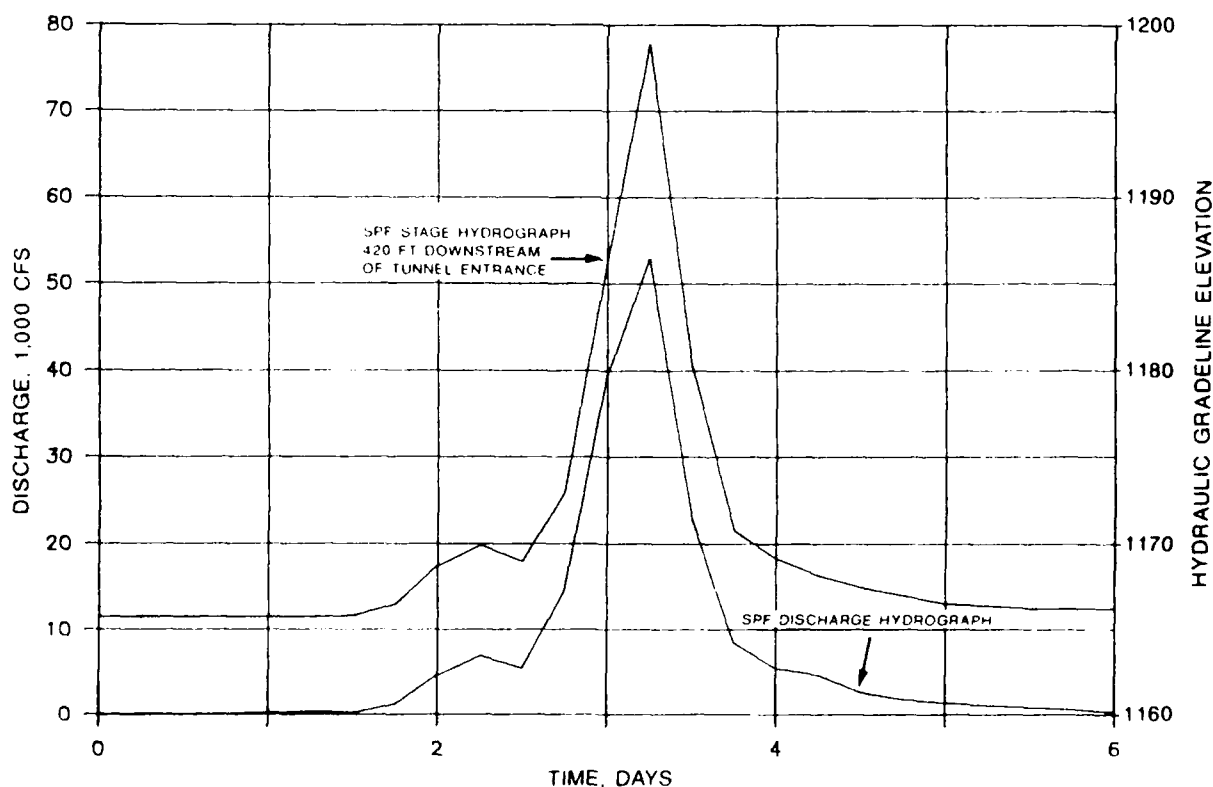


Figure 5. SPF discharge and stage hydrographs

during two instances when upstream "bridge failures" were simulated by dumping a specific volume (137,500 ft<sup>3</sup>) of debris into the model at once. These failures occurred approximately 1 hour prior to the peak and during peak discharge. While numerous scenarios of both debris inflow and the inflowing hydrograph could have been tested, the specific conditions selected were assumed to be representative and tested on types 4-7 design approaches to evaluate the relative differences between designs. Debris tests were not conducted on design types 1-3.

#### Test sequence

21. The SPF hydrograph, tailwater hydrograph, and debris conditions described in the preceding paragraph were run with types 4, 5, 6, and 7 design approaches. Design performance was evaluated by observation of debris accumulation at the tunnel entrance and the maximum water-surface elevation in the approach area. Time-lapse photographs were taken at 5-min intervals in the model and a video camera recorded the entire debris test. Photos 10-13 taken at approximately 2 hours past the peak SPF discharge during each test show the relative difference in types 4-7 designs, respectively. The elevation lines painted on the model along the diversion structure were used to observe the maximum headwater during the test. The peak headwater elevation and total losses for each test condition are found in Table 3. Further documentation of the debris tests can be found in Martin (1989).

### Hydraulic Losses

#### Theory and equations

22. A portion of the testing program was devoted to the evaluation of the losses at the tunnel entrance. Prior to the model tests, the tunnels were sized assuming an entrance loss coefficient  $K_E$  of 1.02. This coefficient was based on a 30 percent blockage and determined according to the following equation:

$$K_E = 0.5 \left( \frac{A_T}{\%OPEN \times A_T} \right)^2 \quad (1)$$

where

$$A_T = \text{tunnel area} = 922.7 \text{ ft}^2$$

$$\%OPEN = 70 \text{ percent (30 percent blockage assumed)}$$

The sum of the loss coefficients  $K_T$  is applied to the velocity head at the exit to determine the total head loss  $H_T$  from downstream to upstream for tunnels flowing full with outlet control as follows:

$$H_T = (K_E + K_F + K_O) \times \frac{v^2}{2g} \quad (2)$$

where

$K_F$  = loss due to friction =  $(29.1 n^2 L)/R^{4/3}$  where  $n$  = Manning's coefficient, 0.018;  $L$  = average length of tunnels, 2,100 ft; and  $R$  = hydraulic radius, 8.03 ft

$K_O$  = outlet loss coefficient = 1 (assumed)

$v$  = exit velocity =  $Q/A = 52,800/(4 \times 922.7) = 14.3$  fps

$g$  = gravitational constant =  $32.16$  ft/sec<sup>2</sup>

and

$$K_T = K_E + K_F + K_O \quad (3)$$

During assumed design conditions, when  $K_F$  equals 1.23 and  $K_E$  equals 1.02,  $H_T$  is equal to 10.3 ft. Adding 10.3 ft to the design tailwater elevation of 1195.6 gives a headwater elevation of 1205.9. While the tunnels were sized based on 30 percent blockage, the elevation of the diversion structure was raised to accommodate headwater conditions for a 50 percent blockage factor.

23. Equation 2 assumes no velocity head in the upstream approach when in actuality a velocity head does exist. To calculate the entrance loss coefficient  $K_E$  for the various headwater conditions observed during testing, it becomes necessary to incorporate the upstream velocity head into Equation 2 using the energy equation,

$$\frac{v_1^2}{2g} + HW = TW + \frac{v_2^2}{2g} + \text{losses} \quad (4)$$

where

$v_1$  = 6.5 fps (average velocity upstream of the tunnels, taken from model data)

HW = headwater elevation observed during testing

TW = tailwater = 1195.6

$v_2$  = exit velocity = 14.3 fps

If  $\frac{v_2^2}{2g} + \text{losses}$  is replaced with the right side of Equation 2, the following is obtained:

$$H_W + \frac{v_1^2}{2g} = T_W + (K_E + K_F + K_O) \times \frac{v_2^2}{2g} \quad (5)$$

Defining  $H_T$  as  $H_W$  minus  $T_W$ , using the friction coefficient  $K_F = 1.18$  for the model, and substituting the values assumed for  $v_1$ ,  $v_2$ , and  $K_O$ , Equation 5 can be solved for  $K_E$ . (The prototype  $n$  value of the plastic in the model is 0.016, which is slightly lower than the assumed  $n$  value of shotcrete, 0.018, resulting in a lower coefficient of friction for use in the model equation.)

$$K_E = 0.3145 H_T - 1.973 \quad (6)$$

#### Losses due to debris

24. Table 3 presents the loss coefficients  $K_E$  and  $K_T$  for observed headwater conditions in the model during debris testing. For comparative purposes the table also contains the estimated (EST) design conditions based on Equations 1 and 2. For all designs, except types 4 and 5 with debris, the actual conditions result in lower headwater elevations than those estimated by the design criteria. Types 6 and 7 resulted in the lowest headwater elevation for conditions with and without debris. A chart for comparing head loss measurements is provided in Figure 6.

#### Losses due to blockage

25. Another set of tests was conducted to determine the head loss associated with various blockage conditions. Five tests were conducted by blocking the upper portion of the tunnels in the type 6 design approach with a solid nonporous cover set at elevations that blocked, respectively, 10, 20, 30, 40, and 50 percent of the cross-sectional area of each tunnel. The results are as follows:

<u>Blockage percent</u>	<u>Headwater El</u>	<u><math>H_T</math>, ft</u>	<u><math>K_E</math></u>	<u><math>K_T</math></u>
10	1202.6	7.0	0.23	2.41
20	1203.0	7.4	0.35	2.53
30	1204.5	8.8	0.80	2.98
40	1208.9	13.2	2.18	4.36
50	1217.0	21.4	4.76	6.94

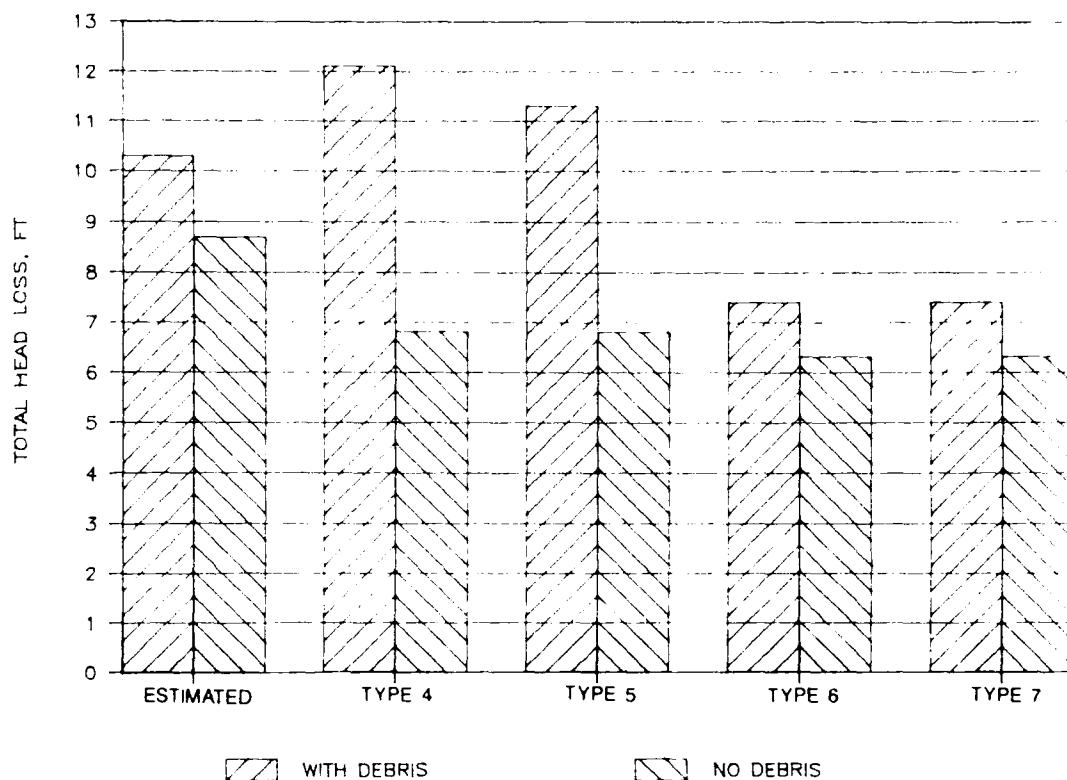


Figure 6. Comparison of head loss conditions

26. Additionally, six tests were conducted by covering the intakes with variable porosity stainless steel screens. These screens had wire diameters and openings which produced, respectively, 23.6, 29.4, 34.4, 39.8, 44.6, and 51.1 percent areal blockages of the tunnel entrances. The results of these tests are as follows:

Blockage Percent	Headwater El	$H_T$ , ft	$K_E$	$K_T$
23.6	1202.8	7.2	0.29	2.47
29.4	1202.8	7.2	0.29	2.47
34.4	1203.8	8.2	0.61	2.79
39.8	1204.0	8.4	0.67	2.85
44.6	1204.8	9.2	0.92	3.10
51.1	1209.0	13.4	2.24	4.42

The plots in Figure 7 show percent blockage versus head loss for porous and nonporous materials.

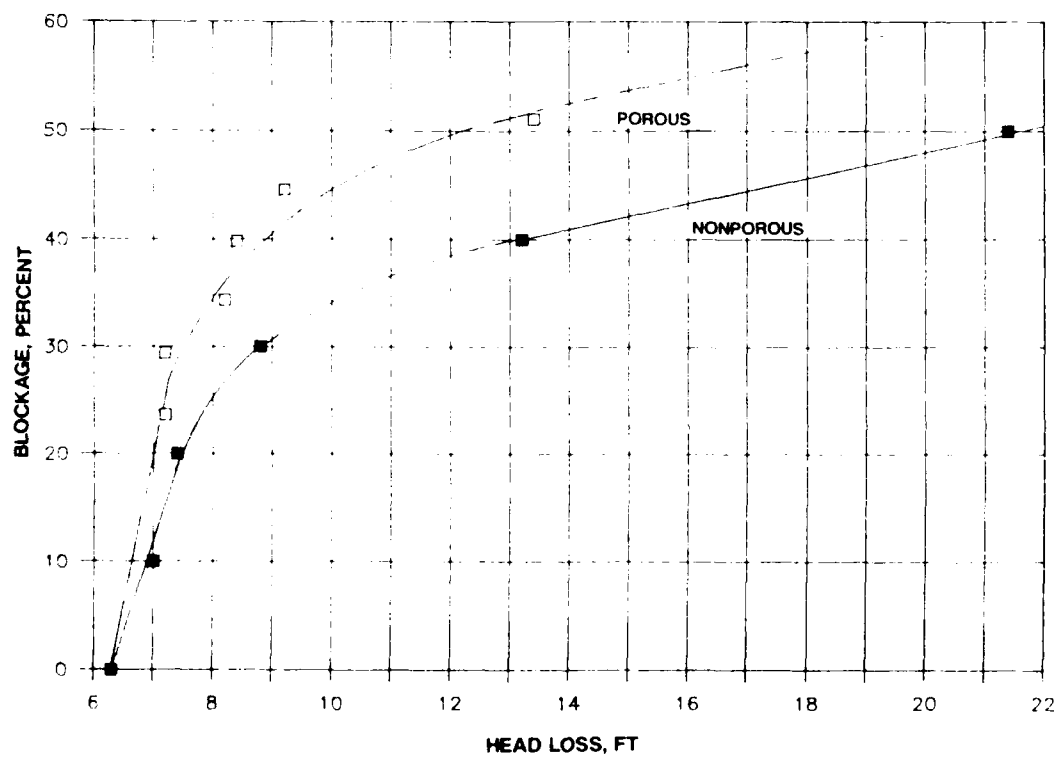


Figure 7. Comparison of head loss for porous and nonporous blockages using the type 6 design approach

#### PART IV: CONCLUSIONS AND RECOMMENDATIONS

27. Results of the physical and numerical model studies led to a number of conclusions. Where possible, all transition sections from the natural channel to the entrance channel that are at or below the design headwater elevation should be curved to prevent flow separation and local eddies from developing (for example, the right wall transition in the type 3 design approach).

28. The 5-ft-radius bellmouth added during the modification for the type 2 design approach eliminated air-entraining vortices and was partially responsible for the headwater elevation lowering from 1203.6 to 1202.5. The 10.5-ft-radius bellmouth first tested on the type 6 design approach was particularly effective in preventing debris accumulation since the radius allowed the debris to turn or pivot into the tunnel. The use of a bellmouthed entrance is therefore recommended whether the type 6 or the type 7 design piers are used.

29. Of the seven designs studied, the type 6 and type 7 design approaches have the most hydraulically efficient pier configuration for passing debris (Figure 8). Both types resulted in maximum headwater elevations of

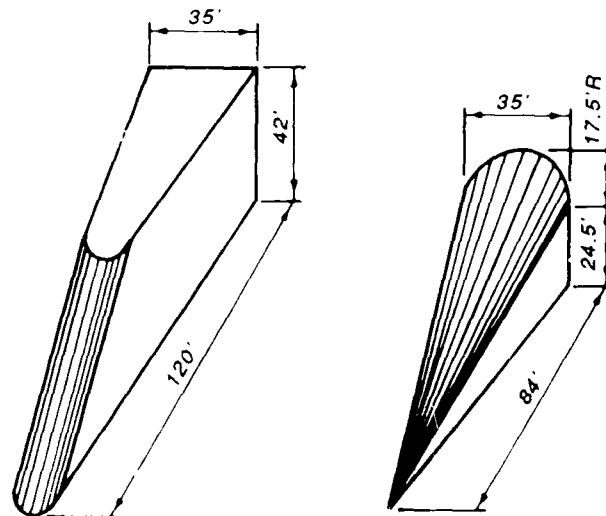


Figure 8. Types 6 and 7 pier dimensions

approximately 1203.0 during the SPF hydrograph with debris. The type 6 design had some debris accumulation at the entrance to Tunnel C, which likely was caused by the protruding bridge pier on tunnel pier 2. The type 7 design had

no debris accumulation at the entrance during the debris hydrograph test.

30. Blunt edges and flat surfaces at or below the water surface tend to cause turbulence and to gather debris. Examples of this were observed in pier and portal face designs in types 1 through 5. The sloping noses with circular shaped sections, found in types 6 and 7, helped prevent debris from lodging by eliminating a "flat" surface for the debris to hang upon and by inducing currents that tended to move the debris toward a tunnel opening.

31. While the debris tests on types 4 and 5 designs did not result in overtopping of the diversion structure, these designs are not recommended unless a more careful debris inventory is conducted and further tests are performed.

32. A field trip was taken in April 1988 to inventory debris in the Clover Fork floodplain. Recommendations pursuant to this investigation include the removal of all housing in the floodplain, and the selective clearing of all trees near the channel bank from the first upstream bridge downstream to the tunnel entrance.

33. The selection of the design approach was based on the ability of the configuration to pass the debris through the tunnels and on downstream. The exit channel should therefore be assessed for its capability to maintain the project flood while passing this debris. A bridge located approximately 0.5 mile downstream of the exit portal is a potential location for the accumulation of this debris. Precautions should be taken to prevent this from occurring.

34. This study presents a topic which is critical to the safe operation and performance of many hydraulic structures, that is, the potential hazards of floating debris. While research studies have been undertaken to assess structural alternatives such as booms and "debris" basins (Perham 1987), further research is still needed in debris transport, particularly quantifying the volume as it relates to distribution and time.

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Table 1  
Summary Comparison of Data

Tunnel	2.5 ft Upstream		45 ft Upstream		90 ft Upstream		Water-Surface El at Diversion Structure
	Average Velocity	Percent Distri- bution	Average Velocity	Percent Distri- bution	Average Velocity	Percent Distri- bution	
Type 1							
A	13.2	27.2	7.0	26.0	N/A	N/A	1203.6
B	12.4	25.5	7.0	26.0	N/A	N/A	
C	12.2	25.1	7.6	28.3	N/A	N/A	
D	10.8	22.2	5.3	19.7	N/A	N/A	
Type 2							
A	12.6	25.0	10.6	24.1	9.2	26.0	1202.5
B	13.2	26.2	11.6	26.4	9.0	25.4	
C	13.5	26.8	12.2	27.7	9.0	25.4	
D	11.1	22.0	9.6	21.8	8.2	23.2	
Type 3							
A	13.2	25.5	11.6	24.7	9.5	26.2	1202.4
B	13.5	26.1	12.2	26.0	9.2	25.3	
C	13.6	26.2	12.7	27.1	9.3	25.6	
D	11.5	22.2	10.4	22.2	8.3	22.9	
Type 4							
A	12.6	24.0	10.8	23.5	9.1	25.6	1202.4
B	13.8	26.2	12.5	27.2	9.2	25.7	
C	13.9	26.6	12.2	26.6	9.3	25.9	
D	12.1	23.2	10.5	22.7	8.1	22.8	
Type 5							
A	12.5	22.4	12.6	21.4	9.7	27.9	1202.4
B	15.2	27.2	16.7	28.3	8.7	24.9	
C	15.2	27.2	15.7	26.6	8.6	24.7	
D	12.9	23.2	14.0	23.7	7.8	22.5	
Type 6							
A	13.1	26.0	9.2	26.4	6.9	23.6	1201.9
B	12.7	25.0	9.1	26.1	7.8	26.7	
C	13.0	25.6	8.1	23.2	7.8	26.5	
D	11.8	23.4	8.5	24.3	6.8	23.2	
Type 7							
A	12.1	23.9	10.8	26.2	10.2	28.5	1201.9
B	13.5	26.6	10.6	25.6	9.1	25.5	
C	13.2	26.1	10.2	24.9	8.3	23.2	
D	11.9	23.4	9.6	23.3	8.2	22.8	

Table 2  
Debris Distribution

<u>Item</u>	<u>Description</u>	<u>Model ft<sup>3</sup></u>	<u>Prototype ft<sup>3</sup></u>
<u>Large Items</u>			
Large trees	35 to 90 ft tall (includes 28 fabricated trees)	14.3	386,100
Subtotal (20% total volume)		14.3	386,100
<u>Small Items</u>			
Small trees	20 to 35 ft tall (includes variable- diameter logs 30 ft long)	20.0	540,000
Branches/twigs	5 to 20 ft tall	20.0	540,000
House/trailer	10 by 10 ft	5.5	148,500
Siding	15 by 10 ft (variable thickness and buoyancy)	12.0	324,000
Subtotal (80% total volume)		57.5	1,552,500
Total		71.8	1,938,600*

\* Total volume is less than 1 percent greater than the estimated volume of 1,925,000 ft<sup>3</sup>.

Table 3  
Entrance Losses

<u>Condition</u>	<u>Headwater Elevation</u>	<u>H<sub>T</sub></u>	<u>K<sub>E</sub></u>	<u>K<sub>T</sub></u>
Type 4 w/debris	1207.7	12.1	1.83	4.01
Type 5 w/debris	1206.9	11.3	1.58	3.76
EST w/debris (30%)	1205.9	10.3	1.02	3.25
EST wo/debris (0%)	1204.3	8.7	0.50	2.73
Type 6 w/debris	1203.0	7.4	0.35	2.54
Type 7 w/debris	1203.0	7.4	0.35	2.54
Type 4 wo/debris	1202.4	6.8	0.17	2.35
Type 5 wo/debris	1202.4	6.8	0.17	2.35
Type 6 wo/debris	1201.9	6.3	0.01	2.19
Type 7 wo/debris	1201.9	6.3	0.01	2.19

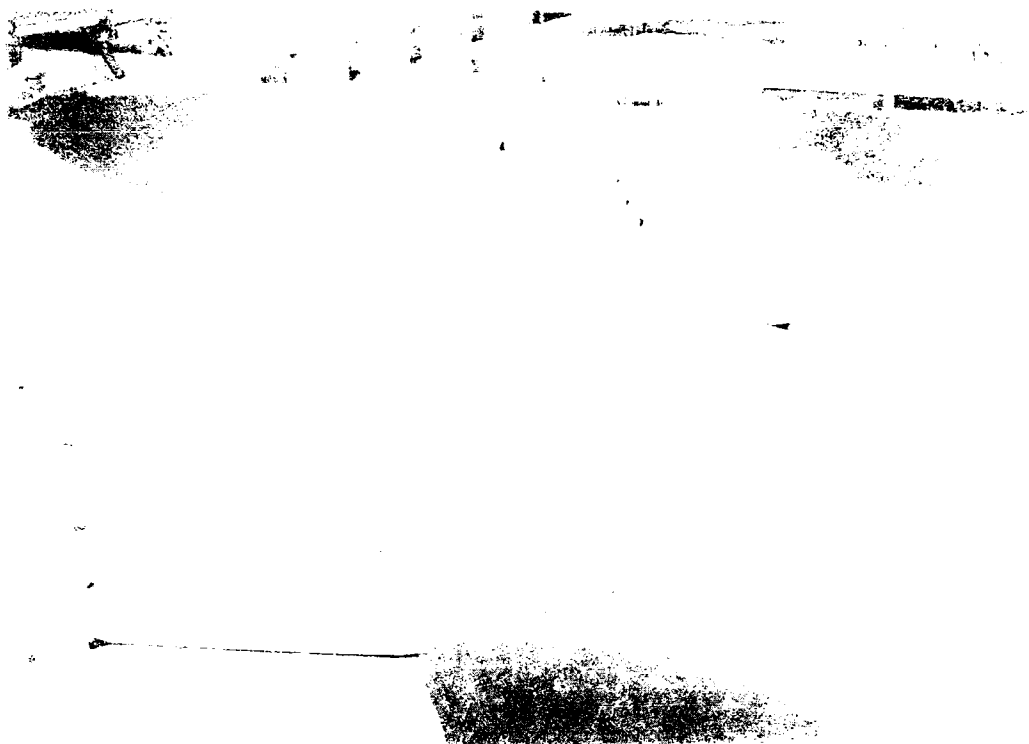


Figure 1. Industrial facility.



Figure 2. Industrial facility with flow direction.

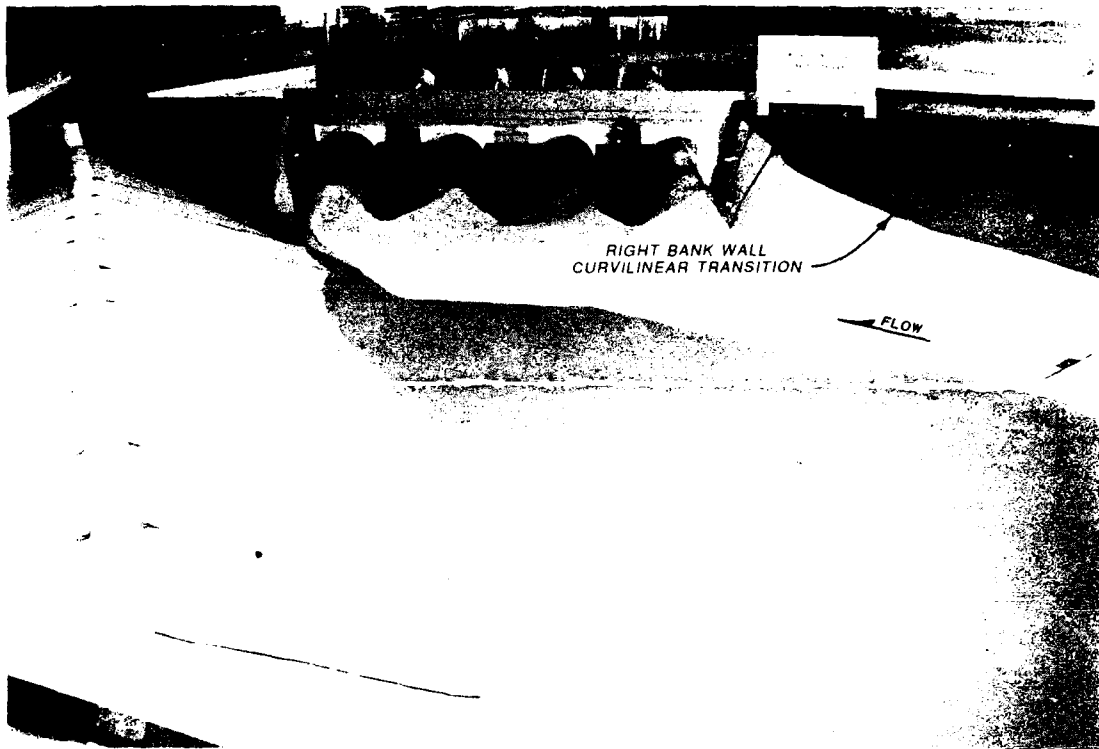


Photo 3. Type 3 design approach

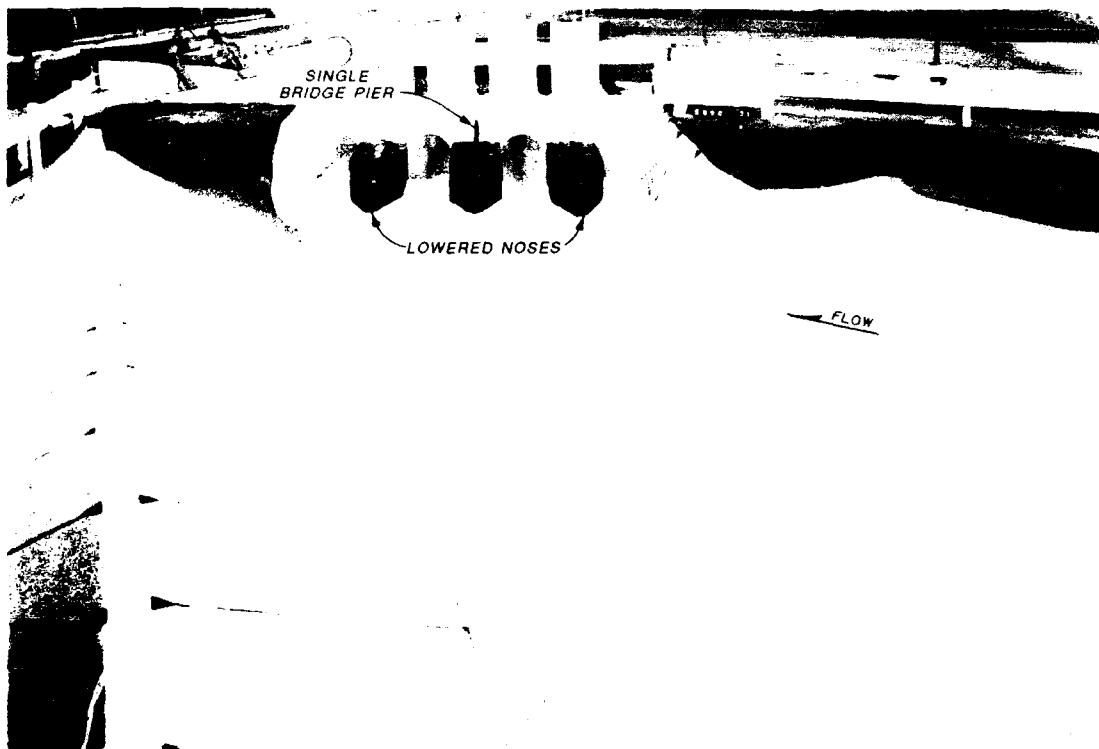


Photo 4. Type 4 design approach

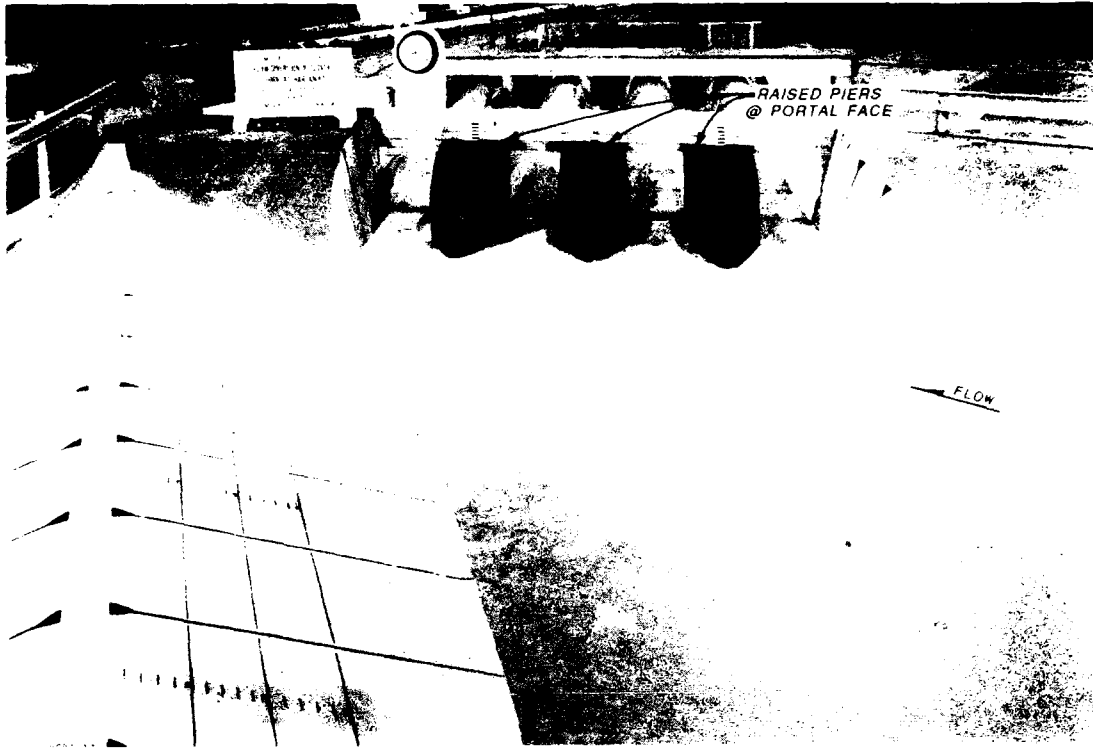


Photo 5. Type 5 design approach

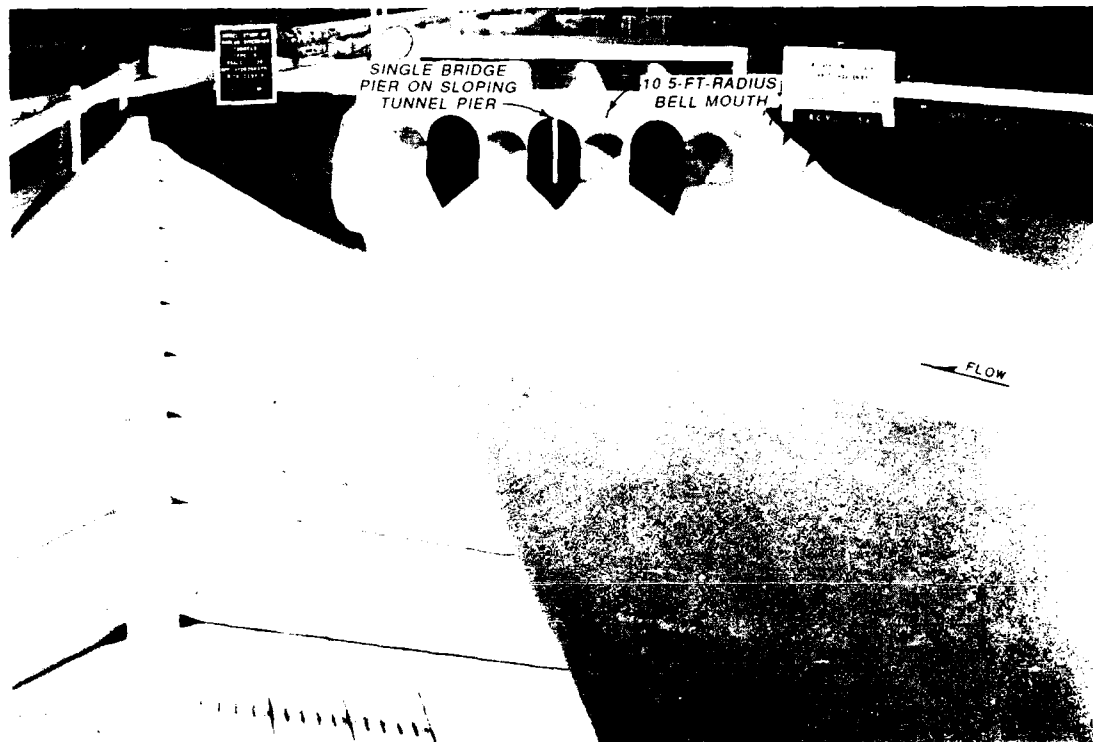


Photo 6. Type 6 design approach



Photo 7. Type 7 design approach

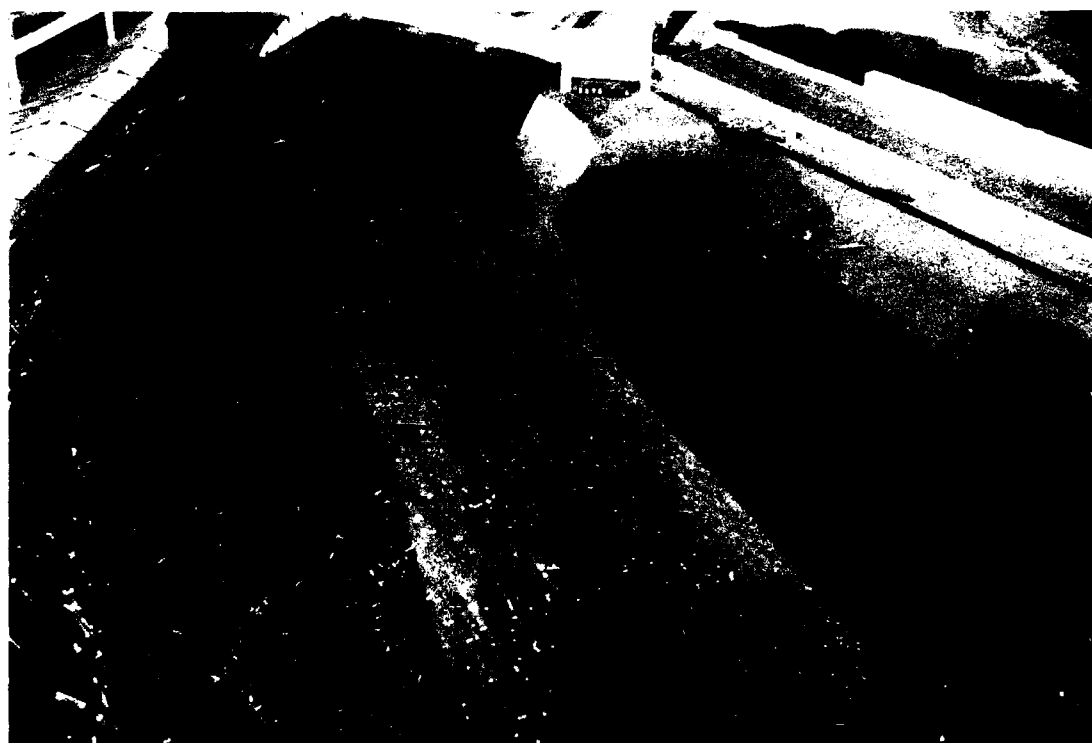


Photo 8. Current patterns for type 1 design approach,  
discharge 52,800 cfs. Confetti streaks reflect  
surface flow patterns



Figure 1. Typical patterns for type 1 debris deposits  
 (a) debris deposits with longitudinal streaks reflect  
 surface flow patterns



Figure 2. Debris field with debris type 1  
 (a) debris deposits with longitudinal streaks reflect  
 surface flow patterns



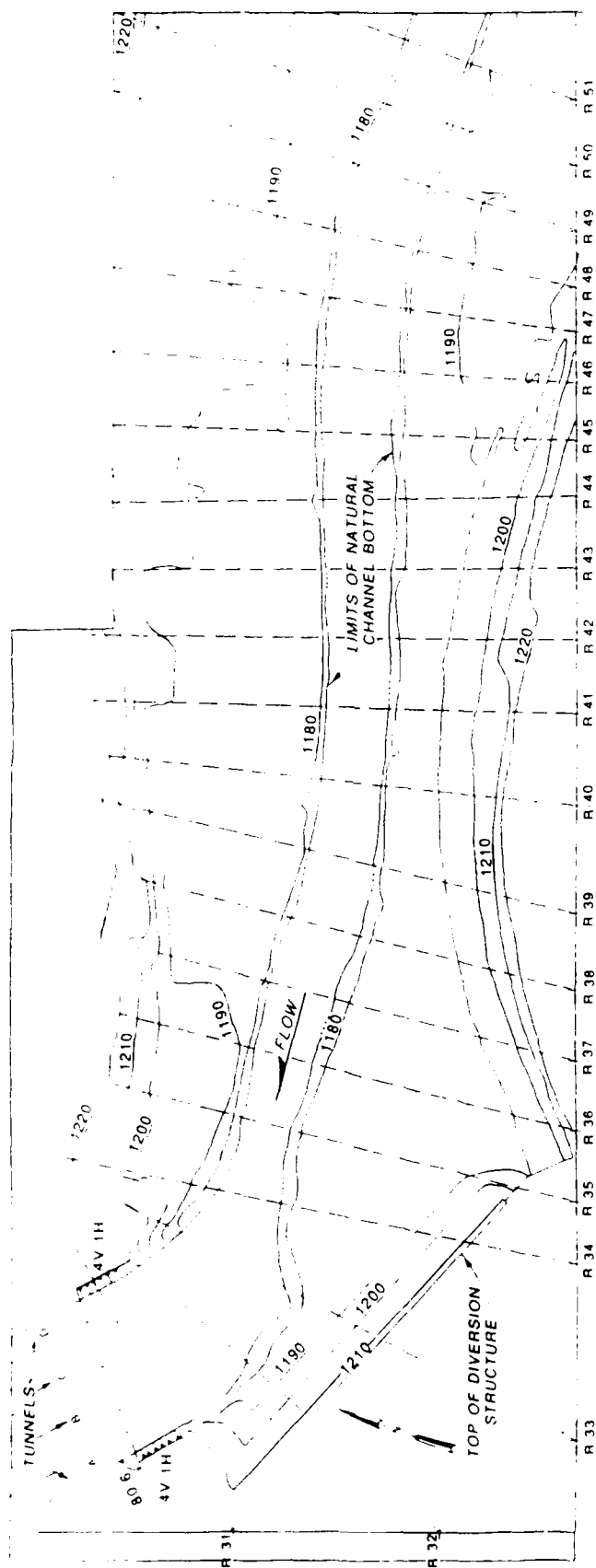
1. Large rectangular structure, possibly a runway or airfield, surrounded by dark, textured terrain.



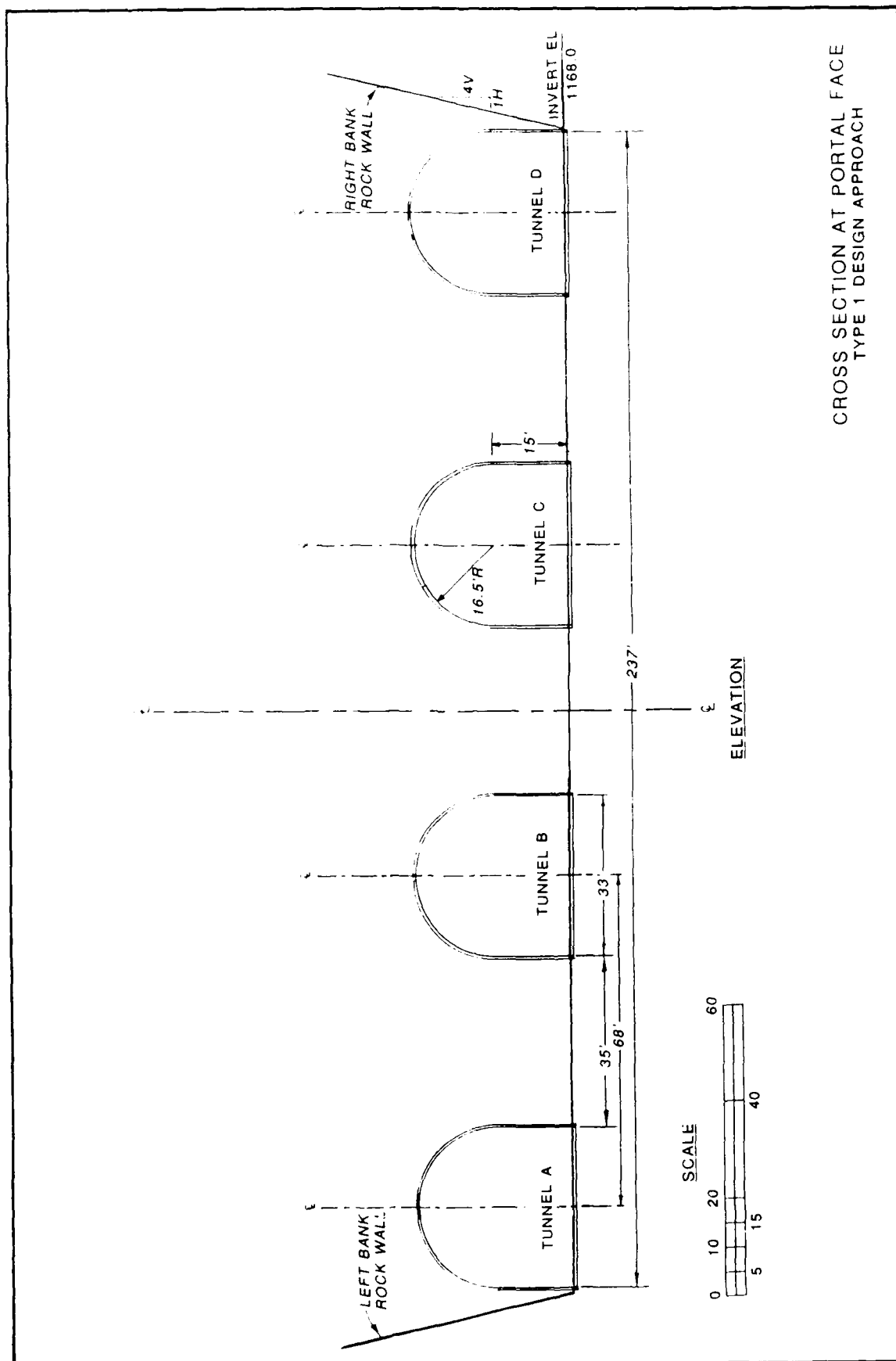
2. Large rectangular structure, possibly a runway or airfield, surrounded by dark, textured terrain.



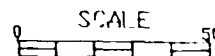
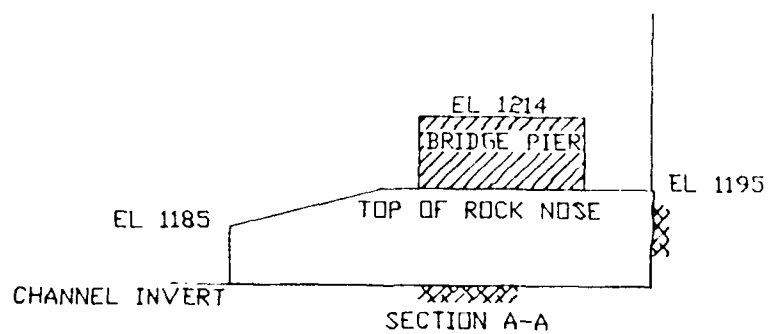
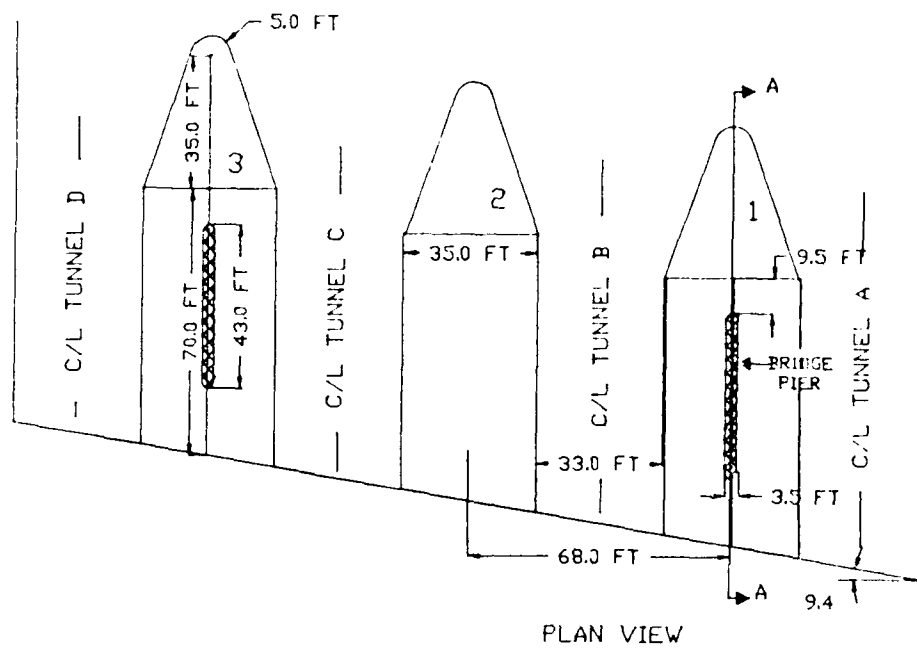
Photo 13. SPF hydrograph with debris, Type 7  
design approach, discharge 48,500 cfs



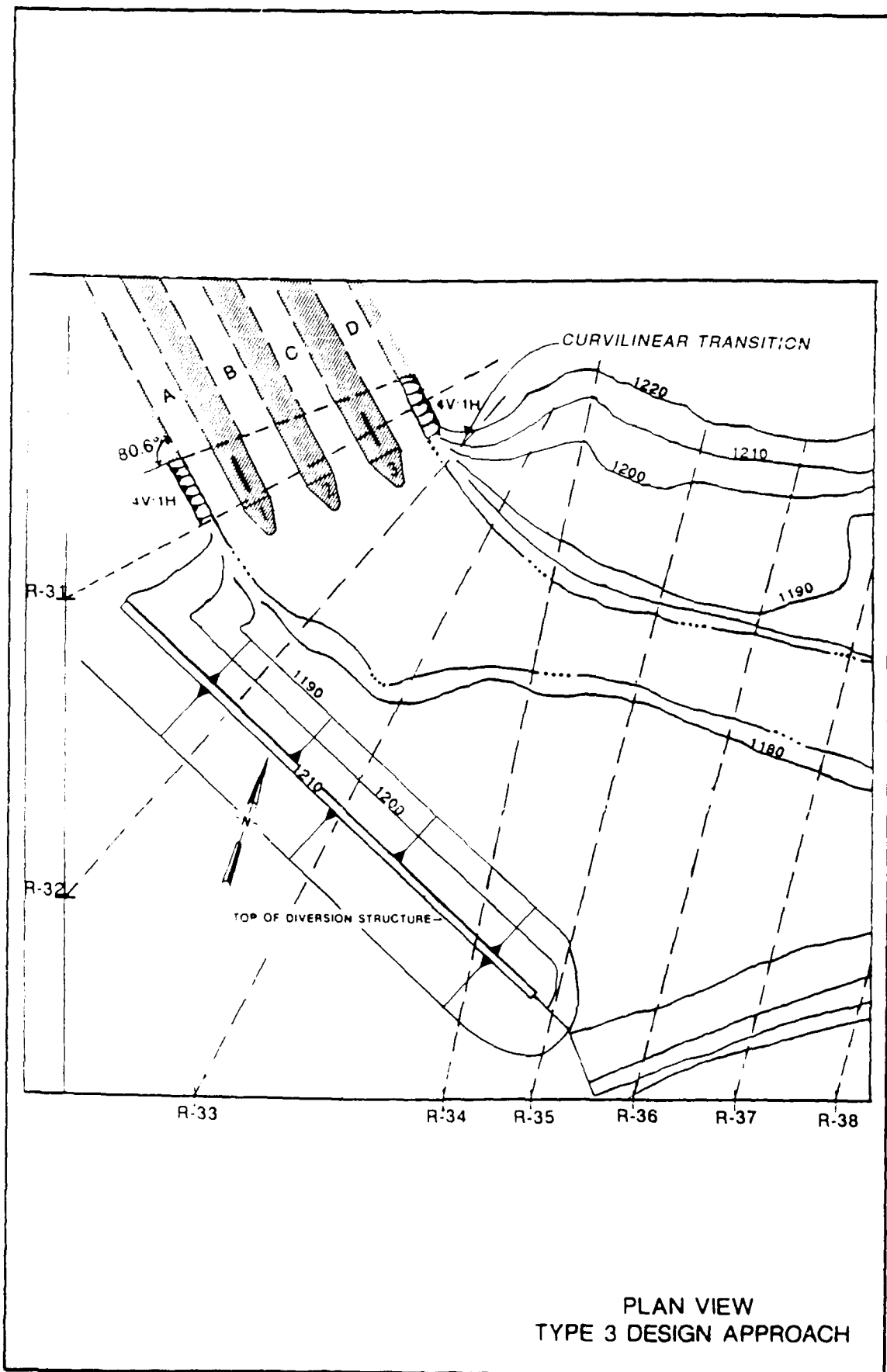
PLAN VIEW  
TYPE 1 DESIGN APPROACH



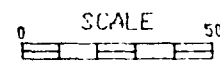
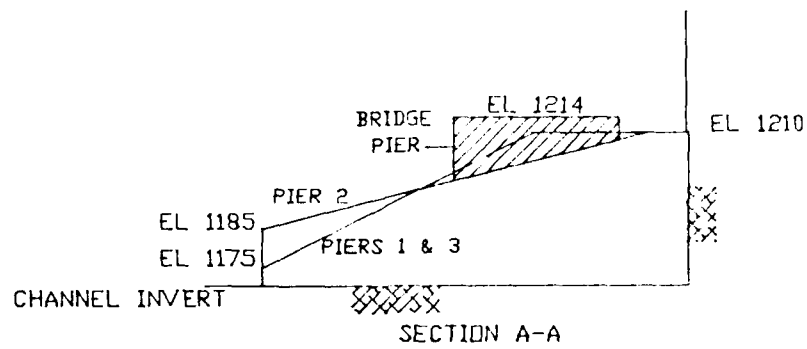
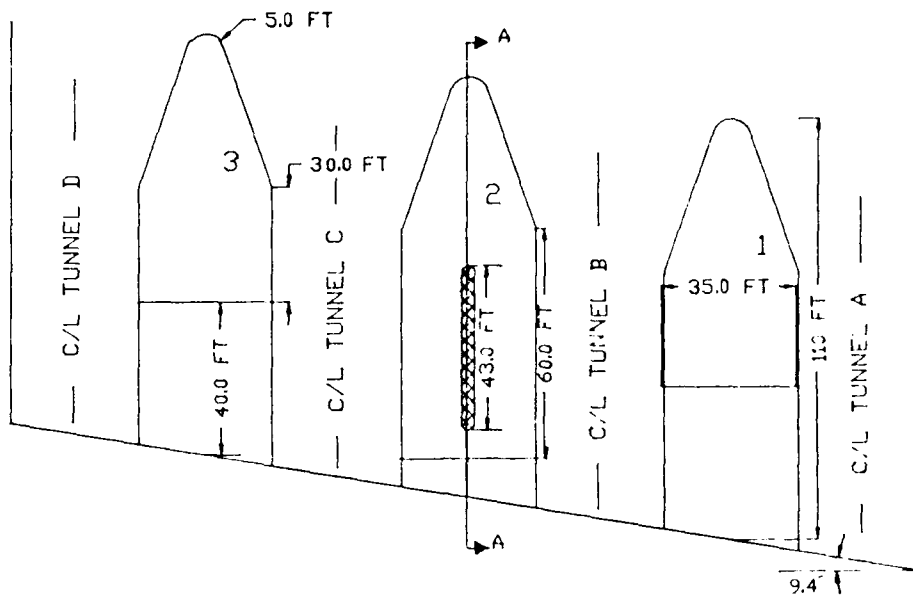
CROSS SECTION AT PORTAL FACE  
TYPE 1 DESIGN APPROACH



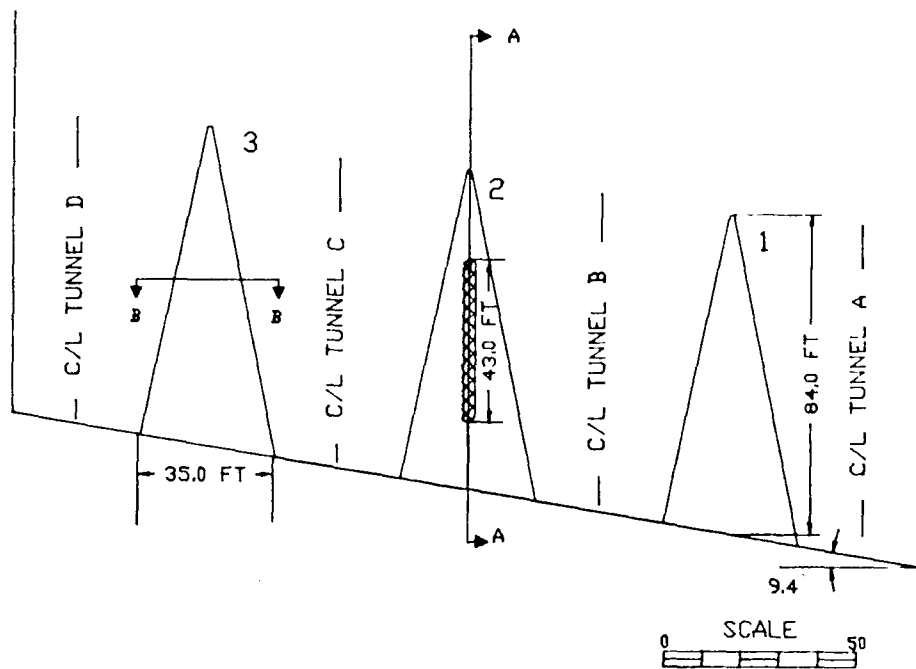
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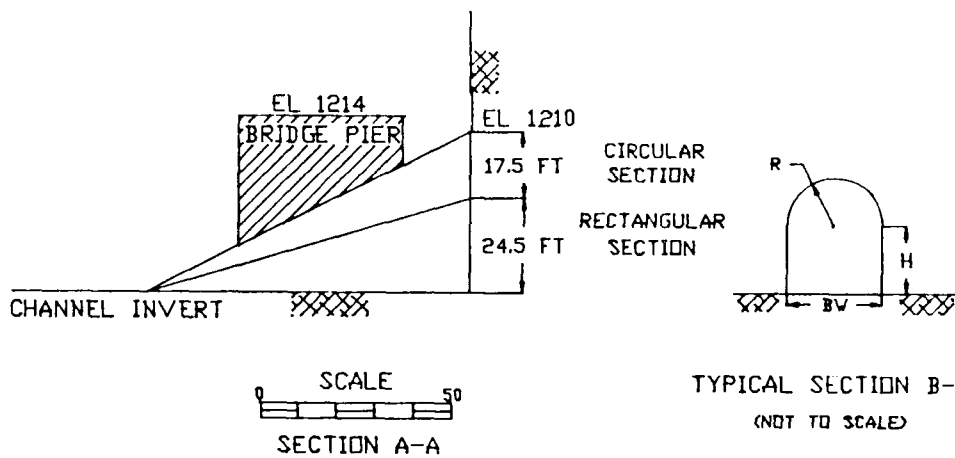




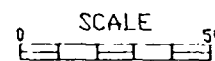
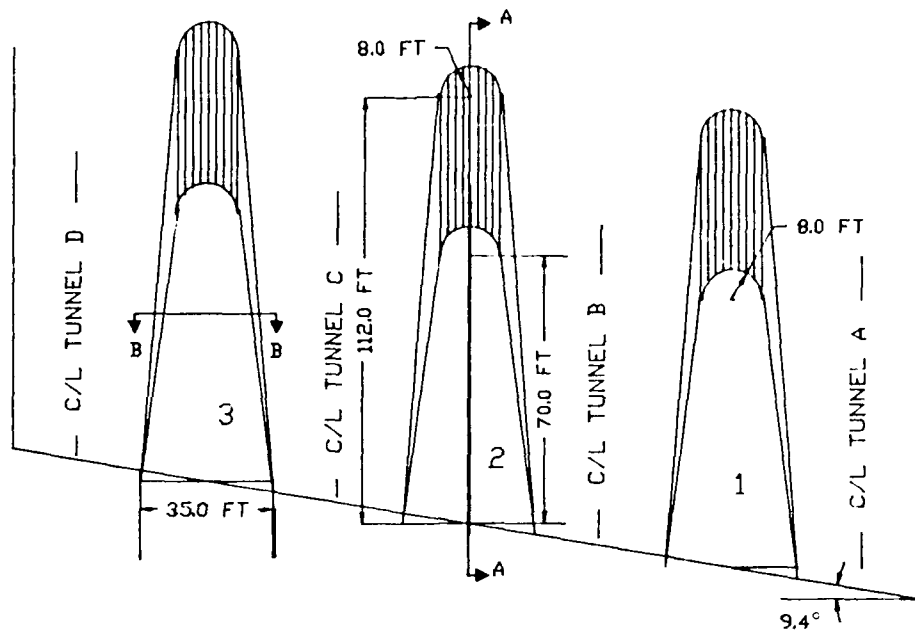
TYPE 5 DESIGN APPROACH



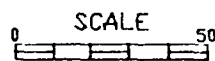
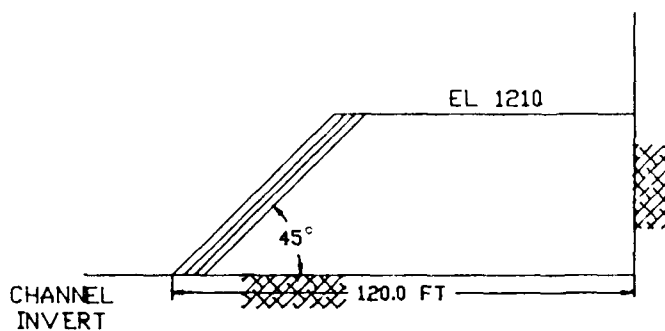
PLAN VIEW



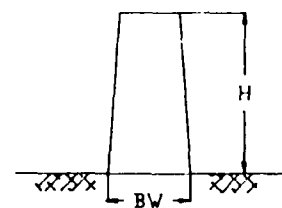
TYPE 6 DESIGN APPROACH



PLAN VIEW



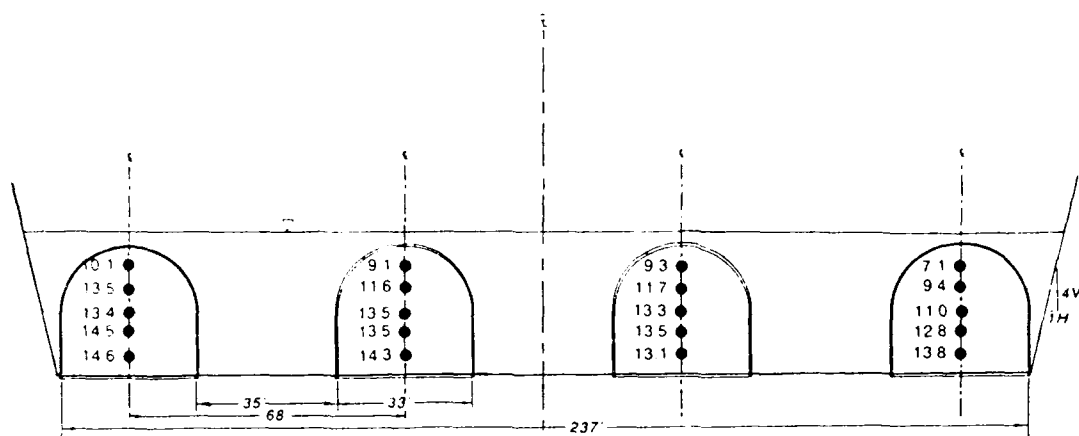
SECTION A-A



TYPICAL SECTION B-B

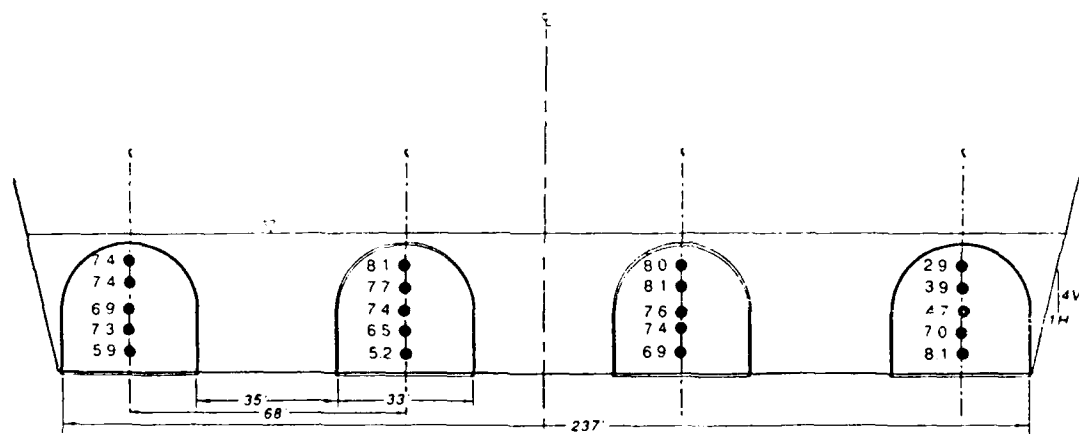
(NOT TO SCALE)

TYPE 7 DESIGN APPROACH

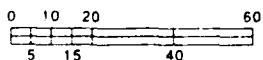


ELEVATION

25 FT UPSTREAM OF PORTAL FACE



SCALE



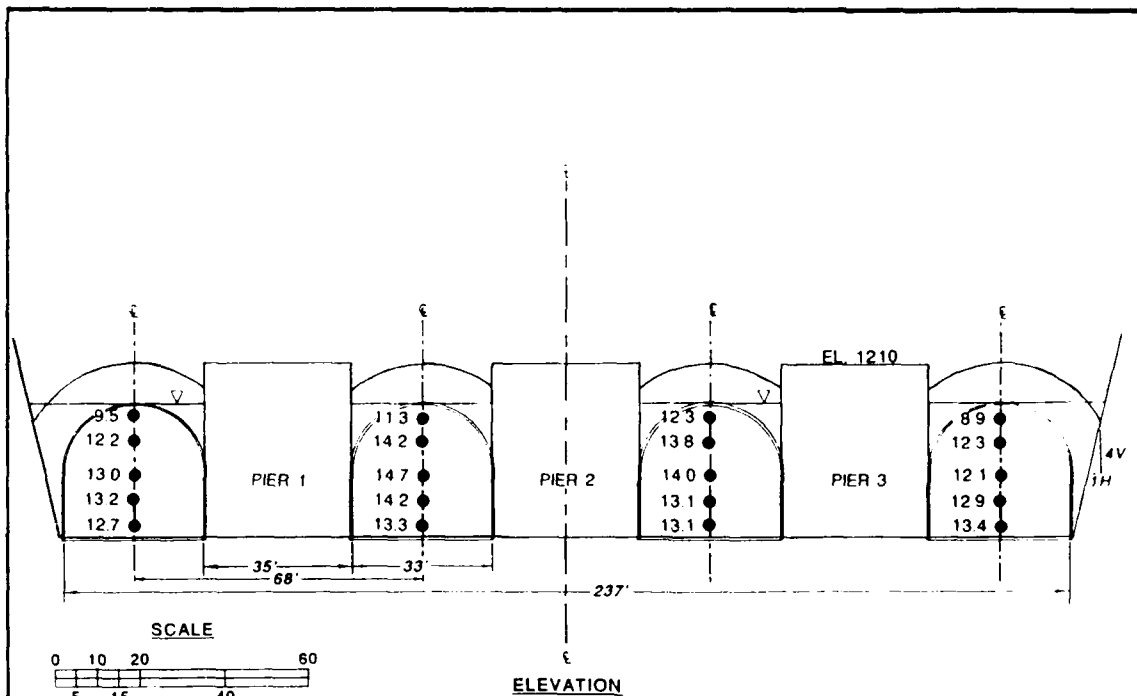
LEGEND

74 ● MAXIMUM VELOCITY, FPS

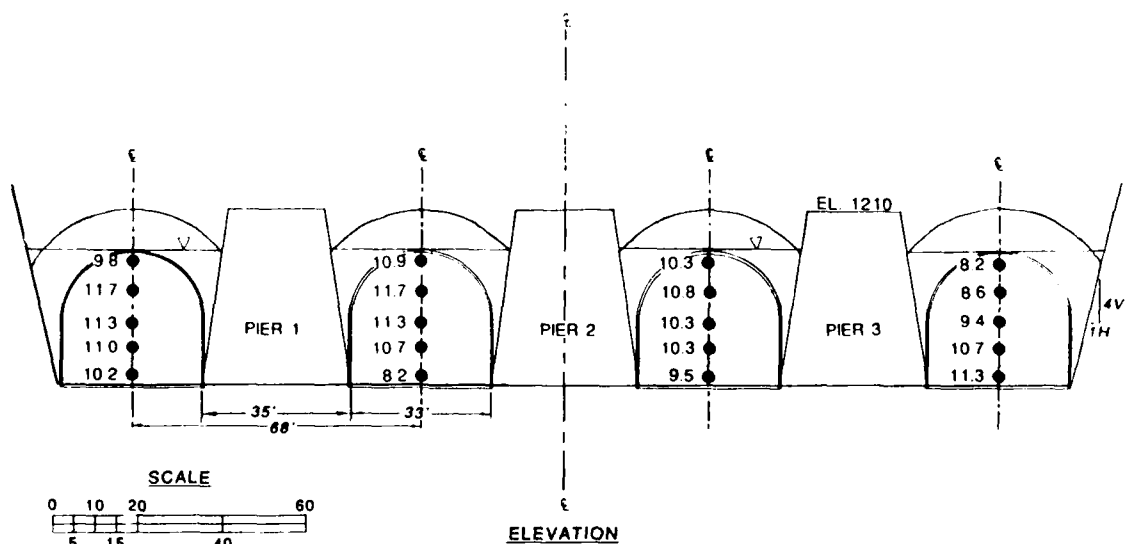
ELEVATION

45 FT UPSTREAM OF PORTAL FACE

MAXIMUM VELOCITIES  
TYPE 1 DESIGN APPROACH  
DISCHARGE 52,800 CFS



2.5 FT UPSTREAM OF PORTAL FACE

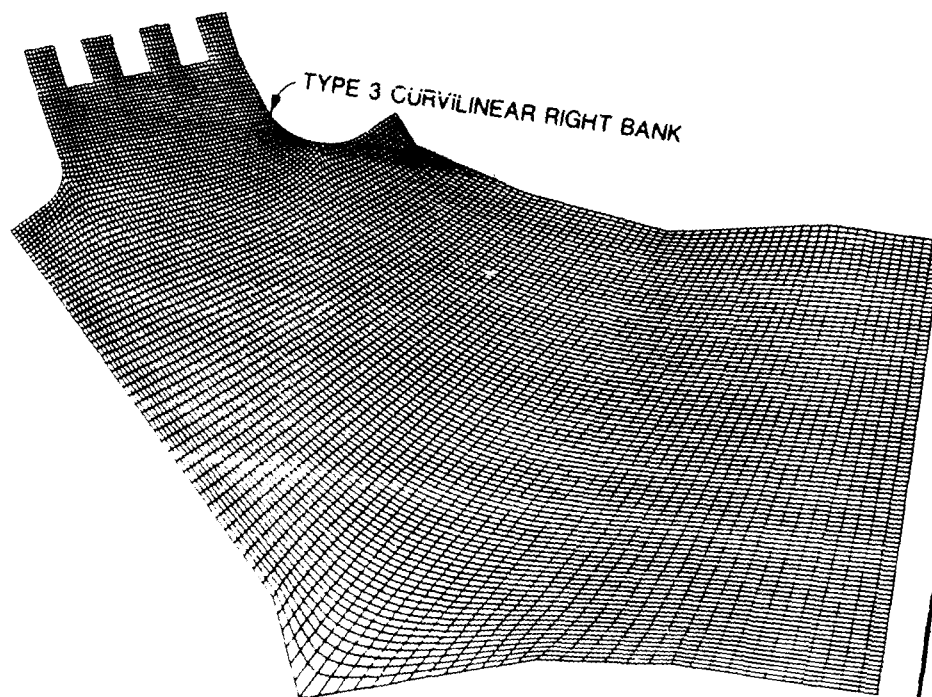
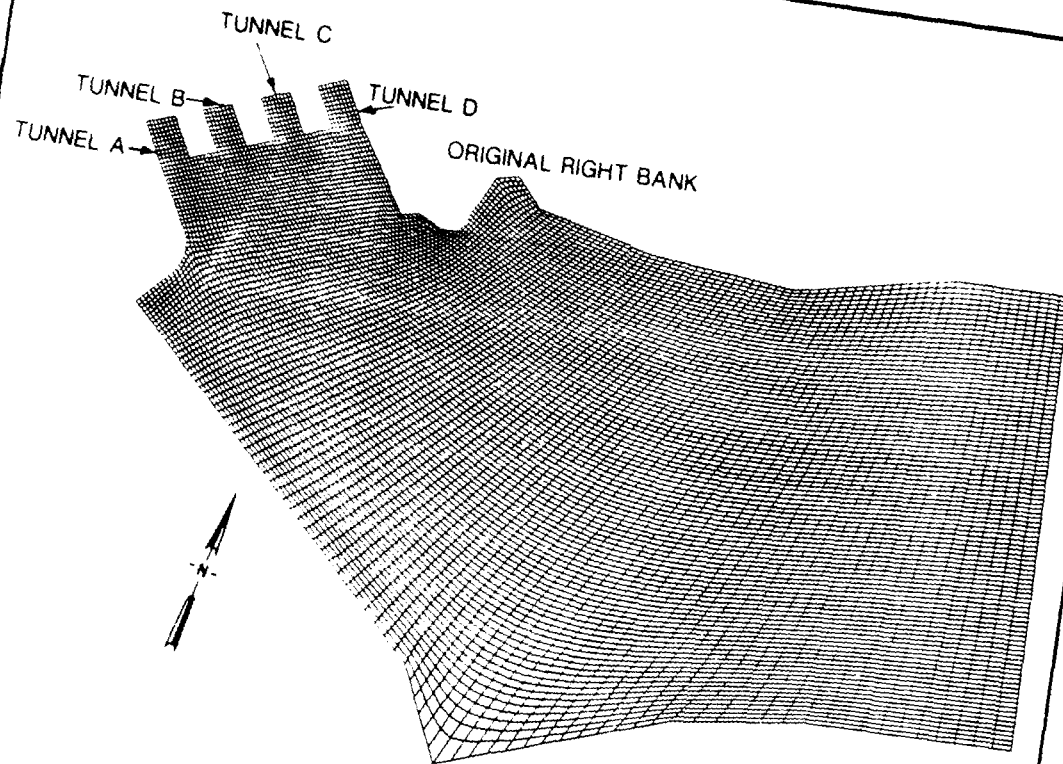


45 FT UPSTREAM OF PORTAL FACE

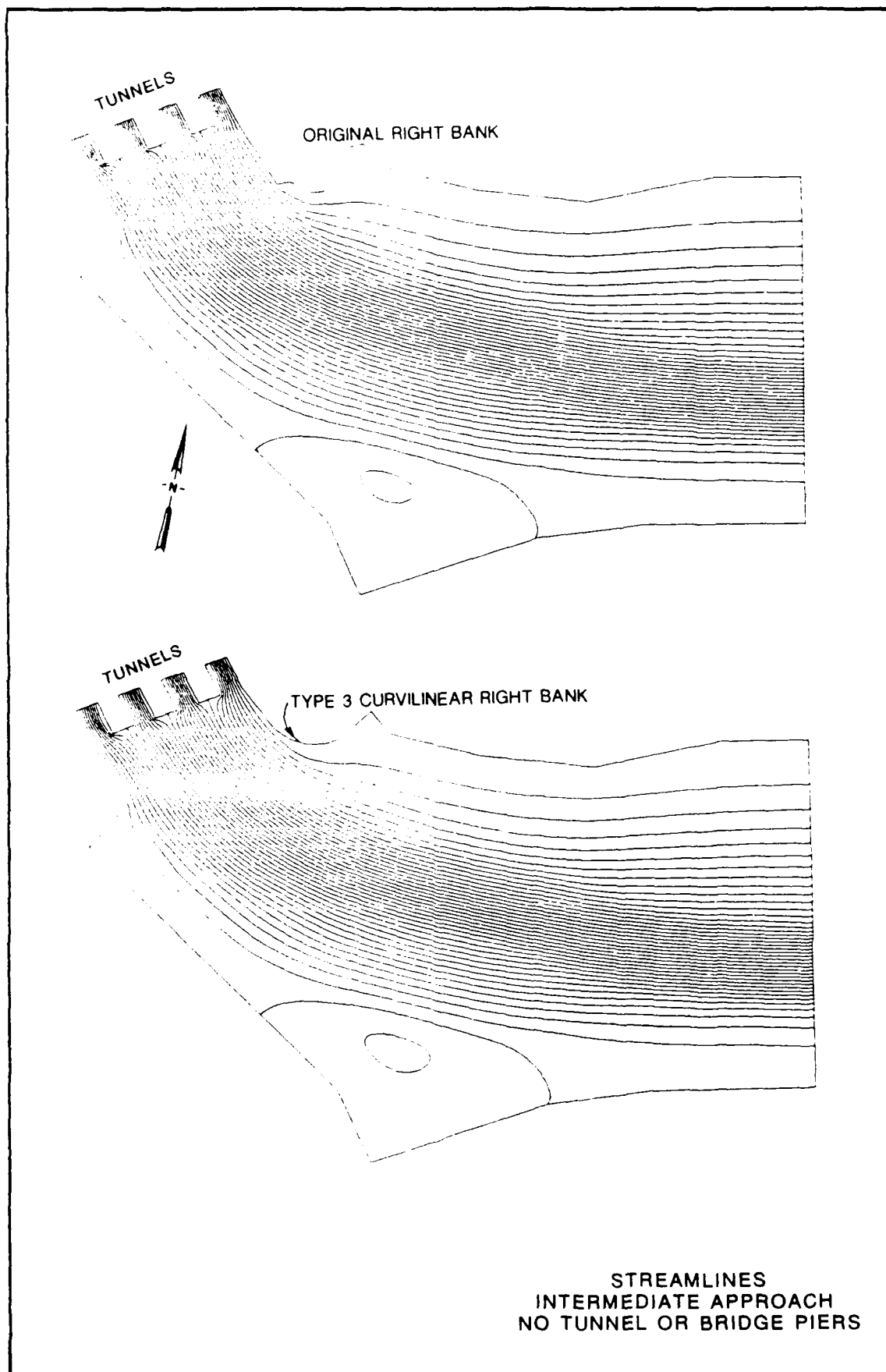
**LEGEND**

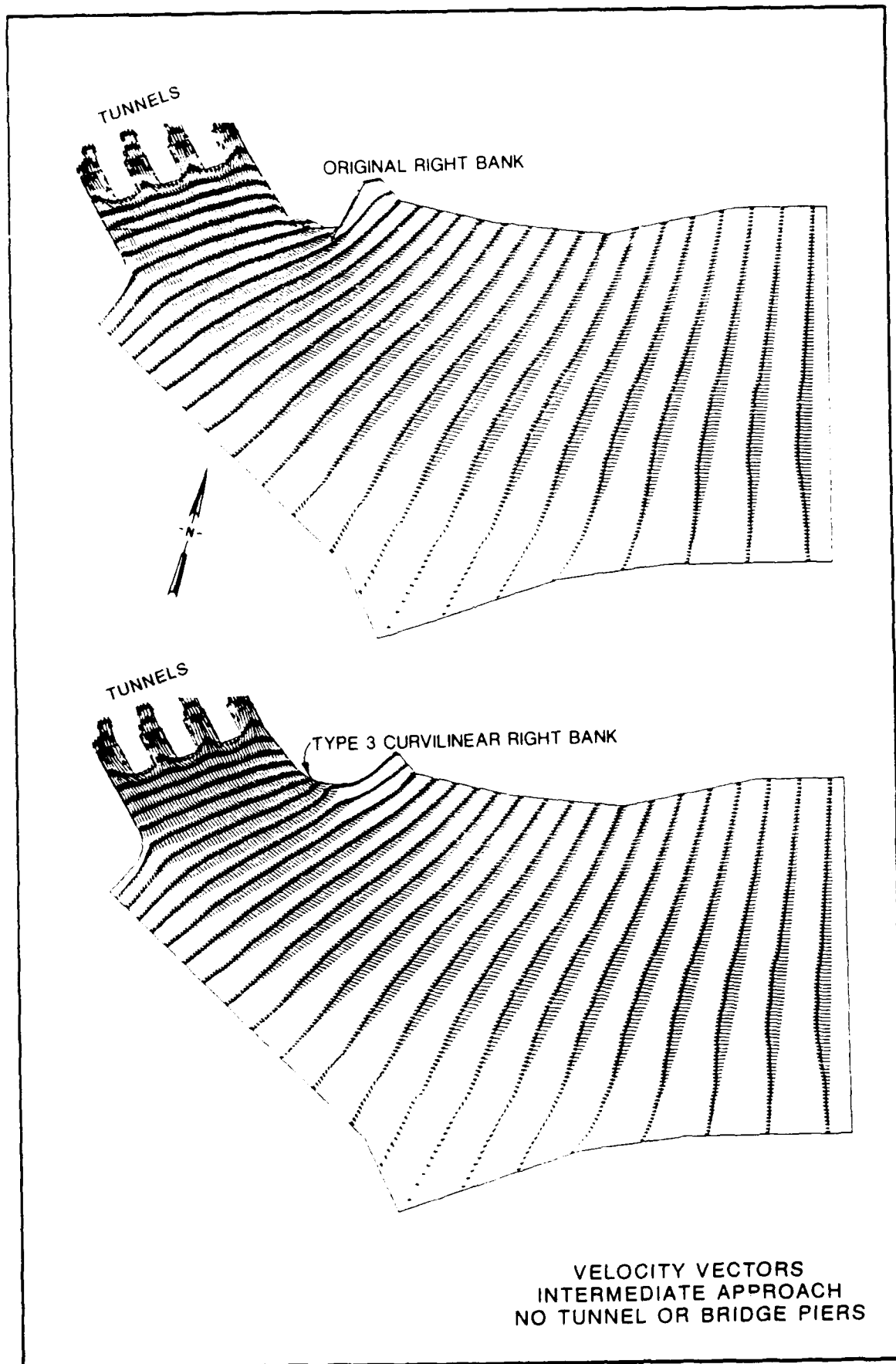
98 ● MAXIMUM VELOCITY, FPS

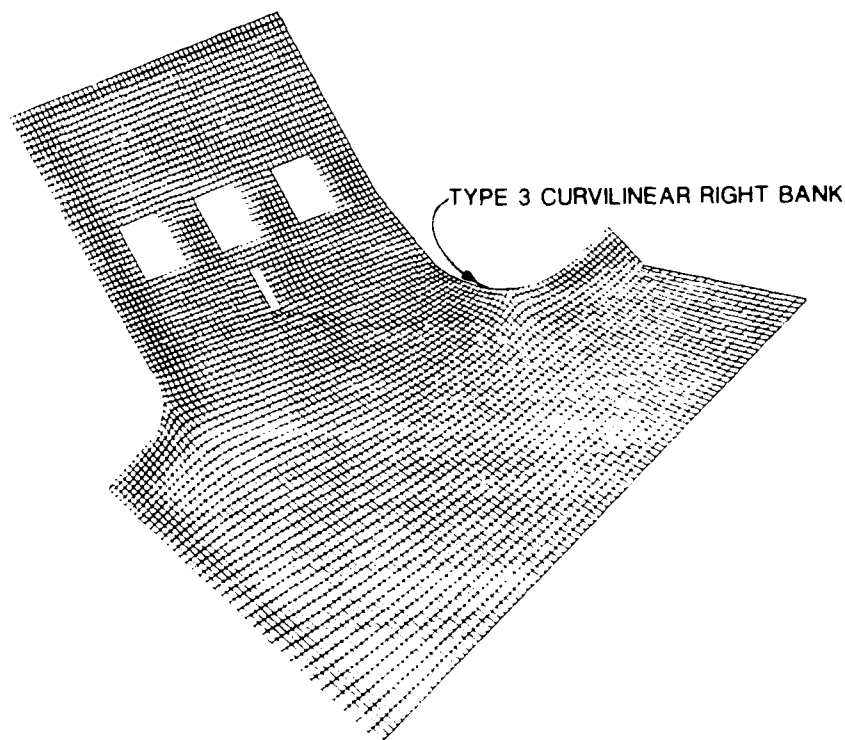
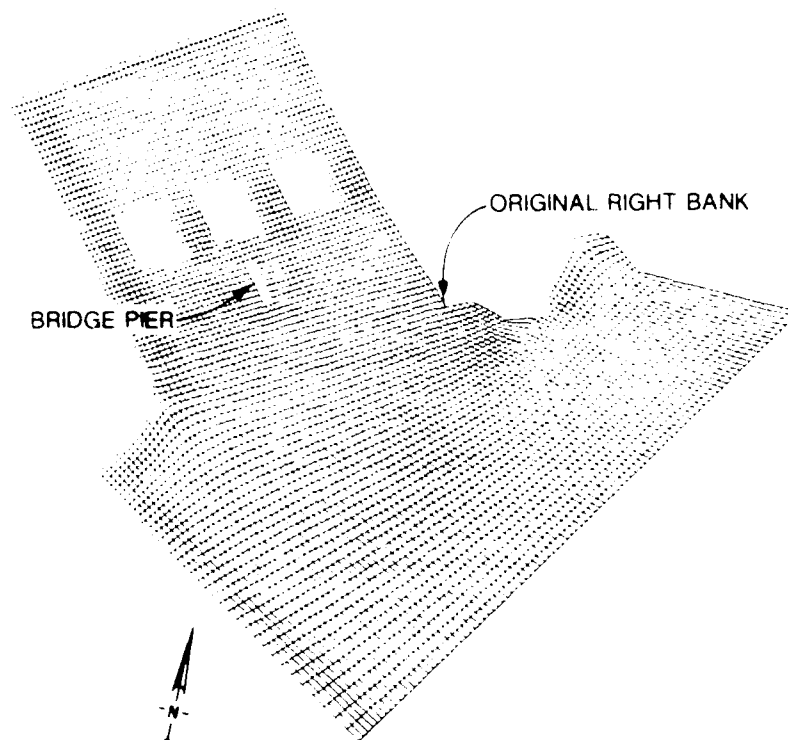
MAXIMUM VELOCITIES  
TYPE 7 DESIGN APPROACH  
DISCHARGE 52,800 CFS



GRID  
INTERMEDIATE APPROACH  
NO TUNNEL OR BRIDGE PIERS







GRID  
CLOSE APPROACH  
TYPE 4 PIERS

